Elliott Bay Waterfront Recontamination Study

Volume II: Data Evaluation and Remedial Design Recommendations Report

Elliott Bay/Duwamish Restoration Program

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Table of Contents

Table of Contents (continued)

Table of Contents (continued)

Appendices

Tables

Acknowledgements

1.0 INTRODUCTION

1.1 BACKGROUND

The Elliott Bay Waterfront Recontamination Study is being conducted by the Washington Department of Ecology for the Elliott Bay/Duwamish Restoration Program Panel to determine the feasibility of conducting sediment cleanup actions along the Seattle central waterfront. The Panel was created as part of a consent decree issued in settlement of a natural resources damage lawsuit brought by the National Oceanic and Atmospheric Administration (NOAA) against the City of Seattle and the. The focus of the assessment has been on impacts to the biological community in Elliott Bay and the Duwamish River due to contamination of sediments associated with combined sewer overflows and storm drains in Elliott Bay and the lower Duwamish River.

The Panel has been tasked with developing and implementing sediment cleanup and habitat restoration projects in Elliott Bay and the Duwamish River. The Panel has identified various remediation sites in these areas. Several high priority sites are located along the Seattle central waterfront. However, concern was raised that sediment remediation along the waterfront may not be successful due to continuing sources of contamination, and the potential for migration of contaminated sediments from other parts of the Duwamish River and Elliott Bay to areas that are to be remediated. In time, the movement and deposition of contaminated sediments onto remediated areas could result in the recontamination of these areas, thereby undermining the project's success.

To address this issue of potential recontamination, Ecology was asked to conduct the present study. This study focuses on the portion of Elliott Bay along the Seattle central waterfront (Seattle Waterfront) from Pier 46 on the south to Pier 59 on the north (Figure 2). The study involves field investigations as well as data analysis and mathematical modeling. Ecology conducted the field investigations through their Environmental Investigations and Laboratory Services (EILS) branch. Modeling analysis was conducted by the Aura Nova Consultants project team. Oversight of the project was provided by Ecology's Toxics Cleanup Program and the Panel's Sediment Remediation Technical Working Group (Sediment Working Group).

Reference will be made to both Metro and King County Department of Metropolitan Services (KCDMS) throughout this report. Metro will be used in referring to the Municipality of Metropolitan Seattle prior to the 1994 merger of King County and Metro, while the acronym KCDMS will be used for references from 1994 forward.

1.2 PURPOSE AND OBJECTIVE

The goal of the Elliott Bay Waterfront Recontamination Study is to determine if it is feasible for the Panel to undertake sediment remediation projects within the Seattle central waterfront by 1997. The overall purpose of the present project is to 1) predict the potential for recontamination of remediated areas from continuing sources and transport of contaminated sediments, and 2) to

estimate the feasibility of conducting successful sediment remediation actions along the Seattle central waterfront.

The specific objectives of the project defined by the Panel and Ecology include:

■ Measure the rate of recontamination and determine the rate of sedimentation in the study area.

■ To the extent possible, identify the components of recontamination and quantify the contribution of each component.

■ If the rate of recontamination is unacceptable, identify source control and/or resuspension control measures that would reduce recontamination to an acceptable rate.

■ Model the impact of these recontamination processes on potential sediment remediation options for the study area.

■ Based on all of the above, provide the Panel recommendations on whether cleanup along the waterfront is feasible, the most appropriate project location(s) for sediment remediation, and the size and type of project(s) that would have the greatest chance of success.

The first two objectives were addressed by Ecology through the project's field activities and are reported in Volume I: Field Investigation Report (Norton and Michelsen, 1995). The last three objectives are addressed in this report.

2.0 DATA REVIEW AND INTERPRETATION

The purpose of the data review is to analyze available data in order to address as many of the project objectives as possible without employing complex modeling. By relying on interpretations of the data with as few assumptions as possible, there will be less uncertainty in the results due to added modeling assumptions.

The data review included interpretation of the data collected by Ecology and others as part of this study, as well as the literature search conducted as part of designing the field study (see Appendix A in Norton and Michelsen, 1995). The data collected for this study include the current meter, and sediment trap chemistry data collected in each quarterly field effort (Norton and Michelsen, 1995), and review of grain size data from the depositional zone and GeoSea grain size studies (McLaren and Ren, 1994). Other data also reviewed for incorporation into the study included:

- Existing data on combined sewer overflows (CSOs), storm drains, and the Duwamish sediment load to characterize sediment sources to the study area (see Section 2.2).

- Sediment chemistry results from sampling along the waterfront by the King County Department of Metropolitan Services and others (EPA, 1988; Metro, 1988; Metro, 1989; Ecology, 1994; Hart-Crowser, 1994) to relate sediment chemistry to depositional and erosional zones determined from the depositional zone and GeoSea grain size studies (McLaren and Ren, 1994).

- NOAA tide data, wave height data, and bathymetric data to determine depth of water at different tide levels and wave heights.

- Current records and physical oceanographic data to determine if other current data and wave height data are available for the study area (Norton and Michelsen, 1995).

- Available sediment trap data and core data from other studies along the waterfront and nearby areas of Elliott Bay, including both sediment trap chemistry and sedimentation rates (Hart Crowser, 1990; EVS & Hart Crowser, 1995).

This review resulted in a characterization of the study area based on sediment depositional pattern, existing sediment quality, sediment trap chemistry, current patterns, and potential sediment sources such as CSOs, storm drains and the Duwamish River. The study area characterization leads to an integrated conceptual interpretation of the observations based on known physical processes acting along the waterfront. These data are reviewed in this chapter, and the conceptual model further developed in Chapters 3 and 4.

2.1 STUDY AREA DESCRIPTION AND SUMMARY OF FIELD ACTIVITIES

2.1.1 Physical Description

The Seattle Waterfront study area is located along the eastern shoreline of Elliott Bay (Figure 1). The waterfront is dominated by a series of piers with intervening slips (Figure 2). The piers are constructed on pilings and are connected to the upland waterfront along the main bulkhead. The bulkhead is constructed primarily of vertical concrete except in the Pier 46/48 slip where the bulkhead is partially asphalt with a small sand beach in the southeast corner exposed at lower tides.

Three City of Seattle CSOs are located along the waterfront at University Street/Pier 56-57, Madison Street/Pier 54, and at S. Washington Street/Pier 48, and one King County Department of Metropolitan Services (KCDMS) CSO at King Street/Pier 47-48. There are five City of Seattle storm drains located at S. Washington Street, Madison Street, Seneca Street, University Street, and Pine Street. There is also a permitted discharge of backwash water from the City of Seattle's Western Avenue Steam plant's water treatment system.

2.1.2 Bathymetry

Initial bathymetry for the Seattle Waterfront (Figure 3) was obtained from existing sediment studies, supplemented with NOAA bathymetric maps of the waterfront. Spot depths at each sediment sampling station along the waterfront were obtained by taking the measured water column depths from the data records and adjusting the depths to the tidal datum of mean lower-low water (MLLW). In order to contour the bottom depths, depths along the bulkhead were estimated based on visual observations. Areas with a visible intertidal zone were given a zero depth; all other locations 4 foot depths. The resulting contours, shown in Figure 3 were compared to previously published NOAA and USGS charts for corroboration. The contours indicate offshore bottom slopes of 0.090 or 1:11 off Pier 56 and flattening toward the south to a slope of 0.050 or 1:20 off Pier 52.

2.1.3 Field Activities

The field activities were conducted by Ecology's Environmental Investigations and Laboratory Services (EILS) Program over a one-year period, October 1993 through October 1994. The major objectives of the field investigations included:

- Characterize chemical concentrations associated with settling particulate matter at selected locations along the central Seattle Waterfront;

- Determine sediment accumulation rates in the study area, including an estimation of net sedimentation (deep burial) and resuspension (gross sedimentation minus net sedimentation);

-4-

■   Estimate current velocity (speed and direction) in the shallower depths in the waterfront area; and

■   Identify sediment transport pathways and areas of deposition and erosion.

The types of instruments and samples collected and sampling frequency from the Ecology field investigations are shown in Table 2-1.

Table 2-1 Field Data Collected by EILS

Sample Type	Number of Stations*	Monitoring Frequency
Sediment Trap	2 Surface, 9 Bottom	Quarterly
Transmissometer	One Vertical Array of 3	Bi-Weekly
Aanderra Current Meter	1 Surface, 5 Bottom	Quarterly
S4 Current Meter	1 Surface, 13 Bottom	Monthly
Sediment Core	3 Gravity Cores	Once

* Surface: ~ 1 meter below the surface
  Bottom: ~ 1 meter above the sea floor

Figure 4 shows a plan view of the instruments and stations. Details of the field sampling and results are presented in Volume I, Field Investigation Report (Norton and Michelsen, 1995).

2.2   POINT SOURCE EVALUATION

2.2.1   Introduction

Potential pollutant sources currently discharging to Elliott Bay in the vicinity of the Seattle Waterfront study area include a number of combined sewer overflows (CSOs) and storm drains (SDs), an industrial discharge, and the Duwamish River. Discharge locations for the CSOs and SDs are shown on Figure 2. Drainage basin areas and system capacities for the CSOs and SDs are summarized in Table A-1 of Appendix A. Maps showing the location of the City of Seattle CSOs (Figure A-1) and SDs (Figure A-2) and Metro CSOs (Figure A-3) and their service areas are also provided in Appendix A.

Contributions from the outfalls and the Duwamish River have been estimated, based on existing data, in order to evaluate whether these sources could recontaminate offshore sediments following remediation efforts. Loads were calculated for the two indicator contaminants of

concern: mercury and polycyclic aromatic hydrocarbons (PAHs). Because this study focuses on recontamination of sediments with contaminated particulates, contaminant loadings were estimated based on the particulate-bound fraction of mercury and PAHs present in the source discharges.

Potential loadings from non-point sources such as fuel spills and leaks, bilge water discharge, and creosoted pilings and bulkheads were not estimated. However, a discussion of their potential impacts can be found in Section 2.5.

2.2.2   Description of Pollutant Sources

Table 2-2 shows the estimated volumes of point source discharges to the Elliott Bay Waterfront area. Table 2-2 includes four discharges outside of the study area: Duwamish River, Denny Way, Vine St. and Connecticut CSOs. The Duwamish River was included because its sediment load was considered to be a significant potential source to the study area. The Connecticut CSO was included because it is located less than one-half mile south of the study area and just north of the Duwamish River East Waterway, and was not included as part of the Duwamish River loading estimate. The Denny Way CSO was included for comparison since it is the largest SD or CSO discharge in Elliott Bay. The Vine St. CSO was included to complete the list of point sources to the Elliott Bay Waterfront area.

The Denny Way and Vine St. CSOs are located about one mile and one-half mile north of the study area, respectively. Neither of these CSOs are considered potential sources of contaminated sediment to the study area since the surface flow is counterclockwise along the Elliott Bay Waterfront (Curl et al., 1987;) which would carry contaminants away from the study area. The area of contaminated sediments associated with the Denny Way CSO has been comprehensively mapped and is limited to within about 1,000 feet of the outfall.

In addition to the above mentioned CSOs outside of the study area, three City of Seattle CSOs (University Street/Pier 56-57; Madison Street/Pier 54; and S. Washington Street/Pier 48) and one King County Department of Metropolitan Services (KCDMS) CSO (S. King Street/Pier 47-48) discharge within the study area (Figure 2). Furthermore, all of the City SDs listed in Table 2-2 discharge within the study area.

To reduce the volume and frequency of overflows to once per year into Elliott Bay, in the early 1990s, the City completed partial separation projects in the University St., Madison St., and S. Washington St. CSO service areas. These separation projects are considered partial because roof runoff was not separated and remains connected to the combined sewer system. As a result of partial separation, overflows from these CSOs have declined from about 4.3 MG/yr to approximately 0.25 MG/yr (Table 2-2). In the future, overflows from the S. King and Vine Street CSOs are also expected to decline as a result of KCDMS's and the City's CSO reduction efforts (see Table 2-2). Current plans call for a reduction of about 22 MG/yr in the S. King St. overflows and approximately 3.2 MG/yr in Vine St. overflows. There will be slight relative

increases in the Denny Way and Connecticut CSO overflows, outside the study area. A CSO reduction project is underway for the Denny Way CSO which will reduce the flows to about 200 MG/yr by 1999.

Table 2-2 Estimated Volumes Discharged to Elliott Bay from Waterfront Sources

Source	Average Discharge (MG/yr)		
	Recent Annual Average	Total During Study Period[e]	Future Annual Average
Duwamish River	343,500	222,500	343,500
Combined Sewer Overflows			
Denny Way (KCDMS)	405 [a]	322	455 [c]
Vine St. (City)	3.3 [b]	2.5	0.1 [d]
University St. (City)	2.8 [b]	0	0.21 [d]
Madison St. (City)	0.7 [b]	0	0.04 [d]
S. Washington St. (City)	0.8 [b]	0	0.01 [d]
S. King St. (KCDMS)	55 [a]	18	33 [c]
Connecticut (KCDMS)	90 [a]	2	93 [c]
Storm Drains[f]			
Pine St.	0.4	0	0.4
University St.	1.7	0	1.7
Seneca St.	0.3	0	0.3
Madison St.	11	3	11
S. Washington St.	5	1.5	5
Industrial Discharges	6.8	7.4	6.8

a) 1981-83 baseline; KCDMS (1995).

b) Brown and Caldwell (1988).

c) 1998 annualized baseline; assumes West Point at 400 million gallons per day; KCDMS (1995).

d) (Chandler, 1995, personal communication); predicted for 1999.

e) October 1993-October 1994.

f) Calculated using TR-55 runoff model.

Five new storm drains, serving approximately 49 acres of downtown commercial area, were

constructed along the waterfront in the early 1990s as part of the city's CSO control projects. Three of these drains (Pine St., University St., Seneca St.) are equipped with low-flow diversion structures which divert runoff from storms smaller than the design one-year storm event to combined sewer system. The one-year storm event is defined as 1.5 inches over 14 hours with peak intensity of 0.3/hr. Therefore, the storm drains begin to overflow at about the one-year event. The CSOs will begin to overflow at flows larger than the one-year event. Based on rainfall records from SeaTac International Airport, it is estimated that approximately 95 percent of the storms are smaller than the one-year event (Ecology, 1992). Consequently, overflows from the separate storm drains and CSOs occur infrequently.

The Madison St. and S. Washington St. storm drains are not equipped with diversion structures, and therefore discharge to Elliott Bay whenever it rains. However, these drains serve relatively small drainage areas (approximately 13 and 7 acres, respectively). Total annual discharge from these drains is estimated at about 16 MG/yr.

The outfall from Seattle Steam's Western Avenue plant is the only industrial NPDES-permitted source currently discharging to the study area. Seattle Steam is permitted to discharge up to 16 MG/yr of backwash water from their ion exchange water treatment system and stormwater. The ion exchange system removes hardness from City of Seattle tap water to reduce scale in their boilers. The backwash water contains particulates from City tap water which accumulates over time in the ion exchange system and must be removed to maintain treatment efficiencies.

Prior to 1981, Seattle Steam was permitted to discharge about 23 MG/yr of stormwater, backwash water, and boiler blow-down water from both their Western Avenue and Post Avenue plants. Discharges declined after 1981 when the Post Avenue plant shut down and the Western Avenue plant diverted boiler blow down water to the combined sewer system. Discharges for the past five years have averaged approximately 7 MG/yr.

2.2.3 Flow Estimates

2.2.3.1 Combined Sewer Overflows

Discharge volumes in this report are derived from direct measurement or are estimated from rainfall-runoff relationships. Rainfall records for the study period were obtained from the National Oceanic and Atmospheric Administration (NOAA) monitoring station at Sand Point. As shown in Table A-4 of Appendix A, rainfall during the study period was below average (approximately 75 percent of the average for the 1951 to 1980 baseline period).

KCMDS measures the volume and frequency of overflow occurrences at all of its CSOs located within the study area. Overflow volumes of the KCMDS CSOs for the 13 months of the study period are given in Table A-2 of Appendix A and summarized in Table 2-2 along with recent and predicted annual average flows based on KCMDS's CSO control plan update (KCMDS, 1995). KCDMS CSOs' recent annual average discharge volumes are based on the 1981-83 baseline

period, and the estimated future volumes are predicted for the annualized baseline year 1998. For KCDMS CSOs during the study period, monitoring records were obtained from KCDMS (Rosenberg, personal communication, 1994).

Discharge records are not available for the City CSOs. Recent and future annual average discharge volumes (Table 2-2) have been estimated based on the results of the modeling conducted as part of the City's CSO control plan (Brown and Caldwell, 1988; Seattle Engineering Department, 1989). Discharge estimates for University St., Madison St. and S. Washington St. CSOs during the study period were calculated based on rainfall records during the study period using the City's design criteria for separation of runoff from paved surfaces which was completed in the early 1990s (Seattle Engineering Department, 1989). Discharge for the study period from the City's Vine St. CSO, which was not part of the previous separation project, was calculated using the recent average annual flow adjusted for the difference in the recent average rainfall and the rainfall during the study period.

Review of hourly rainfall measurements indicated that no storms during the 1993-1994 study period exceeded the 1-year event peak intensity of 0.3 in/hr which would trigger an overflow event for the University St., Madison St. and S. Washington St. CSOs. Therefore, it has been assumed that there was no discharge from these CSOs during the study period.

2.2.3.2 Storm Drains

Stormwater inputs from the central waterfront storm drains have been calculated using the U.S. Soil Conservation Service TR-55 model. Runoff volumes have been determined on a daily basis using rainfall records from the NOAA Sand Point station, which were summed to estimate monthly totals. Storm drains at Pine St., University St., Seneca St. and Madison St. serve predominantly commercial areas in downtown Seattle. For this analysis, the entire basin was assumed to be impervious, and half of the basin area is assumed to drain to the combined sewer to account for the fact that roof runoff was not rerouted to the storm drain system when the drainage systems were separated.

Low-flow diversion structures in the Pine St., University St., and Seneca St. SDs divert runoff from storms smaller than the one-year design storm to the combined sewer. Because rainfall intensities during the study period did not exceed one-year design storm, it is assumed that no discharges from storm drains occurred.

Storm drains at Madison St. and S. Washington St. are not equipped with low-flow diverters and drain directly into the study area. Therefore, runoff is calculated directly using the TR-55 model. The S. Washington St. SD serves approximately 6.8 acres in a mostly industrial area along Alaskan Way. Approximately 75 percent of the basin is assumed to be impervious. Total runoff during the study period was estimated at 8.9 acre-feet from Madison St. and 4.7 acre-feet from S. Washington St.

2.2.3.3 Industrial Sources

Based on discharge monitoring reports, total discharge from the Seattle Steam Western Avenue plant for the 13 month period from October 1993 through October 1994 was approximately 7.4 MG. Average annual discharge for the period 1988 to 1994, which was used for the recent annual average was about 6.8 MG/yr.

2.2.3.4 Duwamish River

The lower approximately 7.5 miles of the Duwamish River is tidally influenced (Dawson and Tilley, 1972). The nearest existing gauging station of river flow upstream of tidal influence is the U.S. Geological Survey Green River gauging station at Auburn. Flow at the Duwamish River mouth was estimated by combining the flow measured at the Auburn station (U.S. Geological Survey, 1994) with estimated storm water and measured KCDMS CSO and estimated City CSO inputs from areas downstream of the Auburn station. Stormwater contributions to CSOs and SDs between Auburn and the mouth of the Duwamish River were estimated based on the drainage area, land use, and rainfall conditions. Land use in areas between Auburn and the mouth of the Duwamish River is summarized in Table A-5 of Appendix A. Surface water runoff from the lower basin has been calculated with the TR-55 model using daily rainfall measurements from the rainfall station at SeaTac International Airport (Table A-4, Appendix A).

KCDMS routinely monitors flow in the nine CSOs that discharge to the Duwamish River between Auburn and the mouth. The city of Seattle monitors overflows in only the largest CSO at Diagonal Way. No overflows were reported at the Diagonal Way CSO during the study period (Mohandessi, personal communication, 1994). For this analysis, it has been assumed that overflows in the other 14 City CSOs that discharge to the Duwamish River below Auburn were negligible.

The measured discharge at Auburn during the study period October 1993 to October 1994 was 213,588 million gallons (MG) compared to an annual average of 317,600 MG for the recent period between 1963 and 1993 (see Table 2-3). Storm water contribution to the Duwamish River during the study period was estimated to be 8,815 MG, and measured CSO contribution was 79 MG for a combined total discharge of 222,500 MG at the mouth of Duwamish River. The recent annual average at the mouth is estimated to be 343,500 MG. Predicted future annual average discharge volumes for the Duwamish River are assumed to be the same as the recent annual average.

The CSO and SD freshwater discharges to the Elliott Bay Waterfront during the study period were estimated to be 356 MG. Existing average annual contributions from the CSO and SD sources are estimated to be about 583 MG with annual average Duwamish River flows at 343,500 MG.

Table 2-3 Annual Loading Summary

SOURCE	DISCHARGE VOLUME (MG/yr) RECENT Average	Min (b)	Max (b)	1993/94 Study Period (e)	FUTURE Average	Pollutant Conc. TSS (mg/L)	Mercury (mg/kg)	PAH (mg/kg)	TSS LOAD (kg/yr) Average	RECENT Min (b)	Max (b)	1993-1994 Study Period (e)	FUTURE Average
Green River at Auburn	317,600	156,200	420,600	213,588	317,600	43	0.052	0.6	5,169,099	2,542,233	6,845,475	3,477,515	5,169,099
Duwamish basin Storm Drains													
Residential	3,115	1,755	4,168	3,347	3,115	1C1	0.20	2.60	1,190,818	670,910	1,593,364	1,279,954	1,190,818
Commercial	15,952	8,987	21,344	4,029	15,952	46	0.23	2	2,777,403	1,564,727	3,716,204	701,675	2,777,403
Industrial	2,506	1,412	3,353	1,014	2,506	59	0.26	8.3	559,627	315,321	748,775	226,574	559,627
Open	3,523	2,039	4,713	425	3,523	1C1	0.1	0.08	1,346,790	779,479	1,801,709	162,581	1,346,790
Metro CSOs (a)	752	293	1,090	79	669	121	1.8	10	344,405	134,190	499,204	36,321	306,392
Seattle CSOs (c)	79	31	114	0.0	0.8	121	0.48	8.5	36,089	14,060	52,348	0	366
Total Duwamish River	343,527	170,717	455,382	222,502	343,366	5.9	0.328	5	7,671,469	3,812,360	10,169,370	4,975,541	7,667,873
Denny Way CSO	405	174	658	322	455	125	2.21	12.1	191,616	82,324	311,316	152,248	215,272
Vine St. CSO	3.3	1.3	4.8	2.5	0.1	121	2.23	21.5	1,511	595	2,198	1,157	46
Pine St. SD	0.4	0.0 (f)	0.5	0.0	0.4	46	0.23	21	63	0	85	0.0	63
Industrial Discharge	6.8	6.5	7.4	7.4	6.8	59	0.26	8.3	1,519	1594	1,612	1,662.4	1,519
University St. CSO	2.8	0.0 (f)	4.1	0.0	0.2	121	0.48	8.5	1,282	0	1,878	0.0	73
University St. SD	1.7	0.0 (f)	2.3	0.0	1.7	46	0.23	21	296	0	400	0.0	296
Seneca St. SD	0.3	0.0 (f)	0.4	0.0	0.3	46	0.23	21	50	0	68	0.0	50
Madison CSO	0.7	0.0 (f)	1.0	0.0	0.04	121	0.48	8.5	321	0	458	0.0	18
Madison St. SD	11	2.0	14	2.9	11	46	0.23	21	1,863	348	2,472	508	1,863
S. Washington St. CSO	0.8	0.0 (f)	1.2	0.0	0.01	121	0.48	8.5	366	0	550	0.0	2.3
S. Washington St. SD	4.6	1.0	6.1	1.5	4.6	59	0.26	8.3	1,027	223	1,362	341	1,027
S. King St. CSO	55	13	80	18	33	121	1.8	10	25,189	5,954	36,639	8,442	15,114
Connecticut CSO	90	2	134	2.0	93	121	0.385	3	41,219	916	61,370	915	42,593
Total CSO & SD study area	84	22	117	30	58				31,976	8,119	45,524	10,953	20,025
TOTAL	344,109	170,916	456,293	222,859	343972				7,937,791	3,904,168	10,589,779	5,140,815	7,945,809

Table 2-3 Annual Loading Summary (continued)

SOURCE	MERCURY LOAD (grams/yr)					PAH LOAD (grams/yr)				
	RECENT Average	Min (b)	Max (b)	1993-1994 Study Period (e)	FUTURE Average	RECENT Average	Min (c)	Max (b)	1993-1994 Study Period (e)	FUTURE Average
Green River at Auburn	269	132	356	181	269	3,101	1,525	4,107	2,087	3,101
Duwamish basin Storm Drains										
Residential	238	134	319	256	238	3,096	1,744	4,143	3,328	3,096
Commercial	639	360	855	161	639	58,325	32,859	78,040	14,735	58,325
Industrial	146	82	195	59	146	4,645	2,617	6,215	1,881	4,345
Open	135	78	180	16	135	108	62	144	13	108
Metro CSOs (a)	620	242	899	65	552	3,444	1,342	4,992	364	3,064
Seattle CSOs ©	17	6.7	25	0.0	0.18	307	120	445	0.0	3.1
Total Duwamish River	2,516	1,250	3,336	1,663	2,515	38,357	19,062	50,847	25,052	38,339
Denny Way CSO	423	182	688	336	476	2,309	992	3,752	1,835	2,594
Vine St. CSO	3.37	1.33	4.9	2.33	0.1	32	13	47	25	1
Pine St. SD	0.01	0.00	0.02	0.00	0.01	1.32	0.00	1.79	0.0	1.32
Industrial Discharge	0.39	0.38	0.42	0.43	0.39	13	12	13	14	13
University St. CSO	0.62	0.00	0.9	0.00	0.04	10.9	0.0	16.0	0.0	0.6
University St. SD	0.07	0.00	0.09	0.00	0.07	6.2	0.0	8.4	0.0	6.22
Seneca St. SD	0.01	0.00	0.02	0.00	0.01	1.1	0.0	1.4	0.0	1.06
Madison CSO	0.15	0.00	0.22	0.00	0.01	2.7	0.0	3.9	0.0	0.2
Madison St. SD	0.43	0.08	0.57	0.12	0.43	39.1	7.3	51.9	0.0	39
S. Washington St. CSO	0.18	0.00	0.26	0.00	0.00	3.1	0.0	4.7	0.0	0.0
S. Washington St. SD	0.27	0.06	0.35	0.09	0.27	8.5	1.9	11.3	2.8	8.5
S. King St CSO	45	10.7	66	15	27	252	60	366	84	151
Connecticut CSO	16	0.4	24	0.35	16	124	3	184	2.7	28
Total CSO & SD study area	47.1	11.2	68.7	15.6	28.2	338	812	478	101	221
TOTAL	3,006	1,445	4,121	2,018	3,036	41,160	20,150	55,309	27,015	41,283

a) CSO volumes based on hydraulic mode of combined sewer system (Swarner, 1995 personal communication; Culp, Wesner, Culp, 1985).

b) Range in annual discharge volumes estimated using minimum rainfall of 20 in/yr and maximum rainfall of 47.5 in/yr (1943-1983 records).

c) Average CSO volumes based on hydraulic model (Brown and Caldwell 1988). Minimum and maximum flows calculated based on Culp, Wesner, Culp, 1985 estimates for Metro CSOs.

d) Overflow from low flow diversion structures estimated assuming that 90 percent of rainfall in an average year is less than the 1-year design storm (Ecology 1992).

e) 13-month study period from October 1993 through October 1994.

f) Assume no storms exceed the one-year event (i.e. no overflows)

- 12 -

These estimates suggest that the Duwamish River contributes more than 99 percent of the total freshwater input to the Elliott Bay Waterfront area. Based on the discharge estimates, the next largest source is the Denny Way CSO which is typically much less than one percent of the Duwamish River flow.

2.2.4 Impacts to the Study Area

Annual estimates of pollutant loads from the Duwamish River, CSOs, and SDs were calculated as the product of the total volume discharged, the total suspended solids (TSS) concentration, and the estimated particulate-bound contaminant concentration in the various sources. Loads were also determined on a monthly basis for the October 1993 through October 1994 study period. Future loads were projected to assess the effects of proposed CSO reduction plans currently being developed by the City and KCDMS.

Particulate fraction source loading has been used rather than the sum of the dissolved and particulate fractions to calculate contaminant loads. This was done in order to focus on particulate loadings to the sediments. Using particulate concentrations and total suspended solids concentrations to estimate potential source loading to local sediments should represent a fairly realistic to conservative estimate for the following reasons.

Particulate loads constitute the majority of total loadings from CSOs and SDs. Previous studies of CSOs and SDs have shown that the particulate fraction ranges from about 65 to 90% of the total metal loading (Tomlinson et al., 1980) and about 90% for the total loadings from aliphatic and aromatic organics (Gavin and Moore, 1982). It has also been shown that re-partitioning between dissolved and particulate fractions following discharge of organic and metal contaminants have a negligible effect on particulate loadings (PTI, 1992; Boatman, 1988). Therefore, the particulate load should represent a realistic estimate of the potential contaminant loading to the sediments.

Also, since the CSOs and SDs are freshwater sources discharging at or near the surface, each discharge plume would remain primarily in the surface waters along with much of its slow-settling finer particulates and all of its dissolved fraction. The surface plume would be carried out of the study area relatively quickly in the stratified surface currents. This would not allow much time for local deposition of the finer particulates from the surface plume. Since all of the TSS particulates are included in the loading estimates, the estimates should yield conservative (i.e. high) loading estimate to local sediments.

Loading calculations were made for the two indicator contaminants of concern in the study area: mercury and total PAHs. These contaminants were chosen because they are the most widespread contaminants above the CSL in the surface sediments along the waterfront, and are considered the most likely to cause recontamination problems.

Pollutant source data were compiled from previous studies. Field sampling to characterize

- 13 -

pollutant sources from CSOs and SDs discharging to the study area was outside the scope of this study and would likely not have been possible, since most CSOs/SDs along the waterfront did not overflow during the study period.

Where possible, site-specific data were used in the loading calculations. However, most CSOs and SDs in the study area have not been sampled. Therefore, available data were used to estimate pollutant contributions from the CSOs and SDs which have not been previously sampled. Most of the existing data on particulate-bound concentrations were from samples collected from depositional areas within the drainage system (e.g. manholes and catch basins). These data are given in Table A-6 of Appendix A.

Because of the limited amount of TSS and particulate concentration data available from the individual waterfront sources, it is difficult to develop an accurate estimates of ranges in contaminant concentrations from those sources. Therefore information on the variability in discharge, which is readily available, was used to estimate the likely range in contaminant loading. This approach produces loading estimates that are representative of long-term contributions to the Seattle Waterfront sediments. Average annual loadings rather than short-term loadings based on episodic events are more appropriate for predicting changes in sediment concentration which can be used in evaluating recontamination potential.

This approach is also supported by available data from the Lander and Michigan CSOs (E.C. Jordan Co., 1984) which show that TSS concentrations vary by less than 30 percent under average wet weather, dry weather, CSO, and first flush flow conditions. The only exception is the dry weather TSS concentration for the Michigan CSO which is about half of the concentration of the other flow periods. Therefore, the average TSS concentrations used in this study, which are primarily collected during storms and CSO events, should yield a reasonable to conservative (i.e. high) estimate of annual solids loadings from the CSOs.

TSS concentrations from storm drains are probably more variable with flow, however there are very few paired flow and TSS data available to determine a relationship. Using geometric mean TSS concentrations for SDs was the only reasonable approach given the available data. Even so, the estimated storm drain TSS loadings to the study area are relatively minor compared with the CSOs, comprising about 10% of the estimated recent annual average and maximum loadings. Therefore, uncertainties in the SD TSS loading estimates will have a relatively minor effect on the overall loading estimate to the study area.

The TSS and particulate concentrations and annual discharge volumes used in the loading calculations are given in Table 2-3. Annual discharge volumes are used along with geometric mean TSS and particulate concentrations to estimate annual loadings. Recent average, minimum, and maximum annual discharge volumes are used to estimate recent average, minimum, and maximum annual loading rates. The minimum and maximum discharge volumes are based on the historical data for annual rainfall and Duwamish discharge, and the rainfall-runoff procedures described in Section 2.2.3. The annual minimum (20.0 in/yr) and maximum

- 14 -

(47.5 in/yr) rainfall used in the calculations were taken directly from the historical minimum and maximum annual rainfall for the period 1943-1983 recorded at the NOAA Sand Point station.

2.2.4.1 Total Suspended Solids (TSS)

Unfortunately, no water quality data is collected along with the flow data from the USGS station at Auburn. Therefore, we could not make flow-weighted TSS loading estimates for the Green River. Instead, the TSS load from the Green River was determined using the geometric mean TSS concentration from 45 samples collected between 1989 and 1993 by Metro at monitoring station A319 near Auburn (Metro 1990; KCDMS, 1994b).

TSS loads from the Seattle central waterfront and the Duwamish basin SDs were calculated based on the geometric mean of TSS concentration data collected from other urban basins in Seattle (Merrill, personal communication, 1989) categorized by land use type (Table 2-3).

TSS loads from the Seattle central waterfront and the Duwamish basin CSOs were estimated using the geometric mean of TSS concentrations from samples collected by Metro as part of the Toxicant Pretreatment Planning Study (TPPS) from the Lander, Michigan, Denny Way, and Hanford CSOs (Cooley et al., 1984). The Denny Way CSO TSS concentration comes from the geometric mean of six recent Denny Way CSO samples collected between March 1992 and April 1993 (Romberg, personal communication, 1994).

One approach to estimating the TSS loading at the Duwamish River mouth is from the sum of Green River and the Duwamish basin CSO and SD loadings (Table 2-3). This yields an estimated recent annual loading of about 11,400,000 kg/yr. This approach assumes that all of the Green River suspended load reaches the mouth. This is probably a conservative estimate since the upper Duwamish requires routine maintenance dredging, so that the loading at the mouth should be less than the sum of Green River and the Duwamish basin CSO and SD TSS loads.

A more realistic estimate of TSS loading from the Duwamish mouth is based on the geometric mean of monthly TSS concentration data from the Metro monitoring station A305 located in the Duwamish River at the south end of Harbor Island near the Spokane Street bridge (Metro 1990; KCDMS, 1994b). As a check, we can see that this loading estimate of about 7,700,000 kg/yr is less than the 11,400,000 kg/yr estimated from sum of the Green River and Duwamish basin inputs, as we would expect.

Monthly TSS loads during the 1993-1994 study period are summarized in Figure 5. Recent annual TSS loads from the waterfront sources under recent average, minimum, and maximum conditions and future conditions are compared in Table 2-3. The data in Table 2-3 demonstrate that the S. King St. CSO at the southern end of the study area is the major existing source of solids discharging into the study area, contributing about 79% of the solids loading. Other potential major sources of TSS come from upstream from the immediate study area, including the Connecticut CSO (41,200 kg/yr), located about 0.4 miles to the south of the study area and

just north of the East Waterway (see Figure A-3 in Appendix A).

To estimate the maximum potential sediment load to the study area contributed by local sources in the immediate study area, the following analysis was made. Assuming that the local sources are deposited in an area 200 meters wide from the S. King St. CSO at Pier 46 to the University St. CSO and storm drain at Pier 57 (800 meters), and using the average net sediment accumulation rate of 0.3 g/cm²/yr measured from the Pb-210 cores (see Section 2.6), the total sediment loading equals approximately 480,000 kg/yr. Based on the average net accumulation rate, the maximum annual average loading from local sources (45,500 kg/yr) corresponds to a sediment contribution of about 9 percent. The remaining 91 percent contribution (434,500 kg/yr) would probably result from some combination of the Connecticut CSO and the Duwamish River plume. Based on the estimated loadings from the Connecticut CSO, with the Duwamish River plume would have to contribute at least 90 percent of the additional TSS loading.

2.2.4.2 Mercury

Duwamish River mercury loads have been estimated using particulate concentration data from samples collected by Metro in 1981 as part of TPPS (Romberg et al., 1984). Ten samples were collected between the turning basin and the south end of Harbor Island. The geometric mean of the ten samples is 0.33 mg/kg (Table 2-3). This value is less than the State's Sediment Management Standards cleanup screening level (CSL) of 0.59 mg/kg and the sediment quality standards (SQS) of 0.41 mg/kg. Other recent measurements of surface sediments in depositional areas near the turning basin, Kellogg Island, and just south of Harbor Island are below the SQS levels for mercury (Teresa Michelsen, personal communication, 1995). Other recent measurements from sediment traps south of Harbor Island (EVS & Hart-Crowser, 1995) show average mercury concentrations of 0.38 mg/kg which are consistent with the 1981 Metro measurements.

These Duwamish River particulate mercury concentrations are similar to concentrations measured on particulates caught in the surface sediment trap (EB1-S) located at station EB1 at the northern edge of the study area, which ranged from 0.25 to 0.41 mg/kg, averaging 0.32 mg/kg over the four seasons sampled. This suggests that the surface trap at EB1 may be influenced by the Duwamish River plume. Other evidence suggesting Duwamish River plume influence for trap EB1-S is discussed in more detail in Section 2.5. Based on the above information, it can be concluded that the Duwamish River is not likely to cause mercury contamination above the CSL in the study area.

Available mercury data for the waterfront sources are limited. Mercury is not routinely analyzed in stormwater samples and has been analyzed in only a limited number of samples from a few Metro CSOs. This is because mercury is not typically considered a significant contaminant in urban runoff as demonstrated in the Nationwide Urban Runoff Program (NURP) (EPA, 1983). The NURP study showed that mercury concentrations were above detection limits in only 9 percent of 128 samples collected in urban runoff nationwide from 28 sites.

Particulate mercury concentrations from the Vine St. and Connecticut CSOs used in the loading calculation (Table 2-3) are from single samples collected from manholes near the outfalls (Tetra Tech, 1988a). The particulate mercury concentration from the Denny Way CSO is estimated from the total and dissolved concentrations of six samples collected during overflow events occurring in 1992-1993 (Romberg, personal communication, 1994). The particulate mercury concentration is calculated as the total mercury concentration (μg/L) divided by the TSS concentration (μg/L). The dissolved mercury concentrations were below detections limits (0.2 μg/L) in all six samples. The geometric mean of the estimated particulate mercury concentration for the Denny Way CSO is 2.21 mg/kg.

Particulate mercury concentrations used to calculate loadings from the other CSOs were taken from the geometric mean for all particulate samples collected from Metro and City CSOs during the Elliott Bay Action Program (Tetra Tech, 1988a). Six Metro CSO samples gave a geometric mean of 1.78 mg/kg, and twelve City CSO samples had a geometric mean of 0.48 mg/kg. As shown in Table 2-3, the data indicate that the concentration of mercury on particulates discharged from CSOs (0.23-2.23 mg/kg) may, at times, exceed the cleanup screening level (0.59 mg/kg).

Data from sediment samples collected from SDs in Seattle, King County, Everett, and Bellingham were used to characterize particulate mercury concentrations in the SDs for this study (Tetra Tech, 1988a; Herrera, 1994; PTI, 1991; KJC, 1987). Available data were grouped according to land use, and the geometric mean of all samples within each land use category (residential, commercial, industrial, and open) were calculated (Table 2-3). Seven samples from SDs draining residential and open areas, 13 from commercial areas, and 19 from industrial areas were used to determine the geometric mean.

Monthly mercury loads for the 1993-1994 study period are presented in Figure 6. Annual mercury loads from the waterfront sources under recent average, minimum, and maximum flow conditions and predicted future conditions are summarized in Table 2-3. The S. King St. CSO, which averages 45 g/yr is the largest local discharge of mercury into the study area, followed by the University St. CSO, which averages 0.62 g/yr.

To estimate the maximum potential for mercury recontamination of the study area from local sources, a similar loadings approach used for TSS was employed. It is assumed that all of the Seattle Waterfront mercury sources (47 g/yr annual average) are deposited in an area 200 meters wide extending from the S. King St. CSO to the University St. CSO and storm drain. Outside source contributions can be estimated assuming they derive from the Duwamish River plume, which contributes over 90 percent of the particulate loading from outside sources to the study area. The Duwamish River plume is estimated to contribute approximately 145 g/yr of mercury, for a total mercury loading of 192 g/yr. Using the average net sediment accumulation rate of 0.3 g/cm^2/yr, the average surface mercury concentration in newly deposited sediments would therefore be about 0.40 mg/kg. Using the maximum annual loading estimate from local sources of 69 g/yr, yields an estimated 0.45 mg/kg concentration in newly deposited surface sediments, which is below CSL, and considerably less than the observed mercury concentrations in the

surface sediments and the sediment traps. This comparison suggests that there is currently no wide-spread mercury impact from the existing local discharges to the study area.

A closer examination of the loading data shows that 45 g/yr of the 47 g/yr discharged within the study area is discharged from the S. King St. CSO (Table 2-3). Lacking wide-spread impact from direct discharges, some local mercury impact might be expected from the S. King St. CSO. The actual area impacted by the S. King St. CSO is unknown. However, if the area impacted was on the order of the size of Piers 46/48 slip (100 by 200 meters), the total mercury load would be about 63 g/yr, including the contribution from Duwamish River plume. This would yield an average concentration of about 1.1 mg/kg in newly deposited surface sediments in the slip. The geometric mean mercury concentration of surface sediment stations in the Pier 46/48 slip is 0.9 mg/kg. To further check this estimate, an examination of the mercury levels in the King St CSO particulates and the surface sediment data in the immediate vicinity of the S. King St. outfall was made.

The average particulate mercury concentration measured from all of the CSOs (Table 2-3) equals 1.3±1.0 mg/kg (± one standard deviation), where the particulate concentration for the S. King St. CSO estimated from the six Metro CSOs (Tetra Tech, 1988a) equals about 1.8 mg/kg (three times the CSL for mercury). However, five of the six surface sediment stations in the slip between Piers 46 and 48 near the King St. CSO outfall show mercury concentrations of less than 1 mg/kg with one higher concentration station (3.3 mg/kg) located in the middle of the slip (see Figure 14). There are two stations at the head of the slip, between the outfall and the high-concentration middle slip station, that have concentrations of 0.39 and 0.9 mg/kg. These stations are located about 150 feet from the outfall. This pattern suggests a correlation with the fine-grained sediments located in the middle of the slip (see Figure 9), as well as limited local impact from the outfall at the head of the slip. The impact of the outfall particulate loading at the head of the slip can also be seen in the lead and zinc surface chemistry data which show higher concentrations limited to the two inner-slip stations (Figures 16 and 17).

To summarize, beyond localized impacts from the S. King St. CSO to the inner Pier 46/48 slip, the other existing sources to the study area do not have the particulate mercury concentrations or loading rates necessary to explain the surface sediment or sediment trap mercury contamination levels observed during the study, and are not likely to cause recontamination above the CSL.

2.2.4.3 Polycyclic Aromatic Hydrocarbons (PAHs)

In addition to the studies described above for mercury, data from a recent study conducted by King County were used to determine the PAH loads from discharges to Elliott Bay. The King County study analyzed contaminants present in sediments collected from a number of catch basins located within the county in residential, commercial, and industrial areas.

Estimated monthly PAH loads for the 1993-1994 study period are shown in Figure 7. Annual PAH loads from waterfront sources under recent average, minimum, and maximum existing

conditions as well as future conditions, are summarized in Table 2-3.

The S. King St. CSO is the largest particulate PAH load within the study area having annual average loadings of 252 g. This may be compared to the next largest source, the Madison St. storm drain contributing an annual average of 39 g. However, in contrast to the local mercury loadings, where the S. King St CSO contributes over 95%, PAH loadings from the S. King St. CSO contribute only about 60% of the average annual PAH loading.

To determine the potential impact of total PAH from the S. King St CSO on the surface sediments in the Pier 46/48 slip, the carbon normalized surface sediment data for LPAH and HPAH from the slip were recalculated as the sum of the un-normalized values for LPAH and HPAH (see Figures 18 and 19). The calculated total PAH surface sediment concentrations in the Pier 46/48 slip have a geometric mean of about 18 mg/kg and ranged from about 8 mg/kg in the outer slip near Pier 46, to 42 mg/kg in the outer slip near Pier 48. Concentrations in the two stations near the head of the slip equaled 12 and 22 mg/kg, respectively. The S. King St CSO particulate concentration of about 10 mg/kg suggests limited influence on the local sediments, and a potential lowering of the ambient total PAH concentration within the slip.

Since local sources other than the S. King St. CSO contribute a significant portion of the PAH loading, it is necessary to estimate the overall impact to the study area of the local discharges. Assuming that all existing Seattle Waterfront total PAH sources (338 g/yr annual average) along with the outside sources (2,170 g/yr) represented by the Duwamish River plume, are deposited in an area 200 meters wide from the S. King St. CSO to the University St. CSO and storm drain, and applying the average net sedimentation accumulation rate of 0.3 $g/cm^2/yr$, assuming 1% TOC, the average maximum total PAH surface sediment concentration equals about 5.2 mg/kg or about 520 mg/kg-TOC. This is well below the CSL for either the LPAHs (780 mg/kg-TOC) or the HPAHs (5,300 mg/kg-TOC).

Even using the maximum estimated annual loading of study area sources (478 g/yr), would result in a surface sediment concentration equal to about 5.5 mg/kg or 550 mg/kg-TOC. Obviously, this is much lower than what is observed either in the surface sediments or in the bottom sediment traps, suggesting that the local existing discharges are not the source of PAHs found in the traps or surface sediments. These values are also below the CSL levels and would not be expected to result in recontamination above the cleanup standards.

2.2.5 Summary

The source loading evaluation has shown that:

1)   Based on the existing sources evaluation, neither the storm drains, CSOs, or the Duwamish River can explain the observed CSL exceedences of mercury and PAHs in the study area's surface sediments and sediment traps.

2)   The Duwamish River dominates all inputs to Elliott Bay in terms of flow and mass
        loadings, but at concentrations that will not cause CSL exceedences in the sediments for
        mercury or PAHs and is not expected to contribute to recontamination.

3)   Outside of the localized impact from the S. King St. CSO to the inner Pier 46/48, the
        other existing sources to the study area are not likely to cause mercury recontamination
        above the CSL.

4)   The existing discharges of PAH to the study area would not be expected to result in
        recontamination above the cleanup standards.

2.3   SEDIMENTOLOGICAL DATA

Two sedimentological studies were performed as part of this project's field effort. Surface
sediments were sampled by KCDMS to determine depositional zones within the study area.
Sediments in the top 2 cm were collected along 15 transects and analyzed for grain size. To
interpret net sediment transport patterns using an analysis of sediment size distribution trends, a
second grain size survey was conducted by GeoSea Consulting covering Elliott Bay and the
Duwamish River up to the turning basin.

2.3.1   KCDMS Grain Size Study

2.3.1.1 Introduction

The following discussion is based primarily on the data generated from KCDMS's sediment
survey (Figure 8). This survey utilized fifteen transect lines designated T1 through T15 oriented
east-west. This data set was augmented with additional data from the Washington Department of
Transportation (WDOT) study of the South Colman Dock (Hart Crowser, 1994), previous
KCDMS studies (Metro, 1984; Metro, 1988; Metro, 1989; Metro, 1993; KCDMS, 1994a), and
Puget Sound Estuary Program (PSEP) studies (EPA, 1988). The percent fines in this discussion
refers to the total of the silt plus clay fractions (i.e., $< 62\mu m$ diameter); the coarse fraction equals
the percent sand plus gravel. Sediment statistics, such as median grain size, sorting (standard
deviation), and skewness, are derived from each grain size sample distribution expressed in phi
units [$\phi = - \log_2$ (grain diameter in millimeters)]. The sediment data were displayed as isopleths
(contours) of percent fines (Figure 9), median grain size (Figure 10), sorting (Figure 11), and
skewness (Figure 12).

The percent fines and sediment size statistics aid in describing and interpreting processes
influencing sediments along the waterfront. In using the data, trends and comparisons are more
significant than absolute values. Percent fines are good indicators of sediment deposition.
Unflocculated or unconsolidated fines are the easiest sediments to resuspend and transport. A
high percentage of fines indicates insufficient currents to resuspend sediments. Conversely, a
high percentage of coarse sediments suggests that the fines have been winnowed or resuspended

and transported to other locations, leaving a coarser fraction of sediments.

Mean grain size covaries with the percent fines and the percent coarse fraction. It serves as an indicator of resuspension and transport mechanisms. The coarser the mean grain size, the more energetic the currents near the bottom. Furthermore, mean grain size trends away from coarse areas toward surrounding fine areas suggest sediment transport in that direction. Sorting is a measure of the uniformity of sediment distribution, where uniform sediments tend either to come from a screened (man-made) source or have been winnowed by waves or currents.

Skewness is an indication of the asymmetry in the sediment grain size distribution curve. Sediments having a component or "tail" along the distribution curve of more coarse grains are negatively skewed and sediments with a component of more fine grains are positively skewed. Skewness is used along with the mean grain size and sorting to infer ongoing transport processes and pathways. For example, a consistent skewness increase directed away from a coarser grained area toward a finer grained area implies the transport of eroded fines from the coarse-grain area to the fine-grain area.

2.3.1.2 Discussion

Percent Fines

Inspection of the composite sediment grain size data and associated statistics yields an interpretation of the sedimentological environments and resuspension and transport processes along the waterfront. The percent fines appears to be one of the better diagnostic indicators of deposition, suggesting that reasonably undisturbed sediments generally occurred in areas where the fines exceeded about 60% of the total sample. The combination of mean grain size, sorting, and skewness augments the interpretation derived from the percent fines.

According to the high percentage of fines (Figure 9), deposition appears to occur under Piers 54 and 56, and under the center of Pier 48. The percent fines also suggests deposition south of Pier 52 and in the center of the Pier 56-57 slip. With the exception of Pier 52, depositional areas with fines ≥ 60% tend to occur in a zone which begins about 300 feet offshore and inside a line connecting the ends of the piers (pier line). Inshore of this apparent depositional zone, the percent fines decrease as a result of wind waves and vessel wakes winnowing the finer materials. In fact, the nearshore stations typically were too coarse-grained to consistently collect a sample with the sediment sampler.

In some inshore areas, either gravel (Pier 51) or sand (Pier 55-57) has been placed near the bulkhead for habitat construction. Through winnowing, the fine materials are deposited further offshore where the sediments typically consist of 58% fine materials. Outside of the pier line, the percent fines trend is less clear because it is masked by the Pier 54-55 cap, and by the influence of ferries docking at Pier 52. Another depositional zone appears to occur at the end of Pier 48, an area where barges often tie up.

A low percentage of fines suggests that erosion occurs under and seaward of Pier 52, at the head end of Pier 48, and at both ends of the Pier 46-48 slip. The low percentage of fines at Pier 52 is related to the recently dismantled northern ferry terminal and indicates scouring from ferry prop wash as ferries used their forward propeller on arrivals to slow and stop, and as they left the terminal on departures.

The lack of fines are also evident at the two sediment caps. The two KCDMS stations located at the southwest corner of Pier 56 containing only 3-5% fines are probably from the Pier 53-55 cap which extended beyond the north boundary (KCDMS, 1994a).

Near the inshore end of Pier 48 lies an intertidal deposit of sand. Swell-like waves converge on the deposit both from the north and the west in the corner formed by two concrete walls. It appears that longshore transport induced by waves transport sand to this local beach deposit, making a well-defined mound. The high percentage of coarse material at the head of Pier 46 may also be explained in part by wave action and littoral drift.

Progressing westward along the slip between Piers 46 and 48, the fines increase in abundance, rising to approximately 70% at approximately the location where the solid bulkhead begins under the container pier. Progressing further to the west the fines decrease to percentages found nearest the shore, equaling 16% at the mouth of the slip. This area is situated just westward of the pier face where the Hanjin container vessels dock. It is probable that the prop wash from the 20-foot propellers on these large container vessels remove the finer sediments leaving the very coarse sediments. In fact, the westernmost station with 16% fines has 25% gravel, a high percentage, possibly indicating scouring by these large vessels.

To summarize:

1)   Nearest shore, in shallower water, the percent fines decrease to a few percent due to wave winnowing and habitat construction;

2)   Outside of the wave-winnowing zone, the percent fines rise to 40-70% if not disturbed by vessel activity and cap placement;

3)   Sediment caps have approximately 0-30% fines;

4)   The long-term effect of prop wash from auto ferries lowers the percent fines to the range of 0-30%;

5)   In the vicinity of the passenger ferries, the percent fines had been lowered from 70% to approximately 60%; and

6)   Container vessels may have lowered the percent fines to approximately 20%.

Grain Size Statistics

The grain size statistics (Figures 10 - 12) tend to confirm the picture developed from the pattern of percent of fines and coarse sediments. Wave winnowing and propeller wash are expected to produce sediments characterized by coarse grain size (less than about 2 phi units), moderate sorting (less than ~ 2 phi units), and negative skewness. Such areas occur along the bulkhead at Piers 52, 57, off Pier 55, and in the vicinity of Pier 48. The opposite characteristics of fine grain size, poor sorting, and positive skewness suggests areas of deposition, little resuspension, or little disturbance. This pattern is observed in a swath from the seaward end of Pier 57 to the landward end of Pier 56, at the seaward end of Pier 54, and in the middle of the slip between Piers 46-48.

An interesting pattern occurs under Pier 57. Here the sediments are poorly sorted medium to fine sand and very positively skewed, indicating a large component of fines. The percent fines under Pier 57 is less than 40 percent suggesting that this area may be receiving recent deposition.

Sediment Caps

Two sediment remediation caps lie within the KCDMS sediment survey area (e.g., see Figure 9). The Pier 51 cap was placed under a portion of the ferry terminal in 1989. The Pier 53-55 cap was constructed in March of 1992.

The KCDMS sediment stations lying along the edge of the Pier 51 cap contain a low percentage of fine sediment (0-26%) with a median grain size of 1 to 2 phi units, a sorting of about 4 phi units, and are positively skewed. This indicates a very poorly sorted, coarse to medium sand containing a component of fines. Without knowing the original condition of the cap, but assuming it began as a well sorted sand, we can speculate that it has become altered by the inclusion of fines. The local gradient in skewness (negative to positive) suggests a direction of transport of fines from the Pier 48/52 slip toward the Pier 51 cap.

The Pier 53-55 cap has an irregular shape and differs in characteristics from northwest to southeast according to the KCDMS sediment data. This material originated from the Duwamish River turning basin and is coarse to fine sand (Metro, 1993). The percent of fines ranges from 1% to 31%, the median grain size ranges from 1 phi unit (coarse sand) in the northwest to 3 phi units (fine sand) in the southeast. This appears to be changed from the post-placement REMOTS camera surveys which showed that the sediments in the southeast portion of the cap were 1 to 2 phi units or medium to coarse sand (Metro, 1993).

Sorting of the Pier 53-55 cap varies from 0 phi units (very well sorted) in the west to 2 phi units (poorly sorted) along the east, to 3 phi (very poorly sorted) in the southeast. Skewness changes from negative in the northwest to positive in the southeast. One interpretation of this data is that fine-grain sediments were deposited during piling removal just south of the cap. Sediment chemistry data shows a strong concentration gradient that is consistent with the piling removal activities (Romberg et al., 1995) (see Sections 2.4 and 2.5).

2.3.2 GeoSea Sediment Transport Study

The GeoSea study (McLaren and Ren, 1994) was designed to examine net sediment transport patterns based on spatial trends in sediment grain size distributions in Elliott Bay and the Duwamish River waterways. More than 500 stations were occupied and 550 grab samples were collected. An attempt was made to incorporate the GeoSea grain size data into the present study area database. Unfortunately the GPS positioning system used by GeoSea was not differentially corrected and generated errors (40-100m) too large to be used in the present study.

The relevant conclusion from the GeoSea study for the nearshore Seattle Waterfront was that no clearly defined distribution pattern was present in this area. This was attributed to " the high degree of anthropogenic disturbance that is present along the docks." According to the GeoSea report, there is a paucity of natural sediment supply to the nearshore areas allowing reasonably accurate delineation of areas showing anthropogenic disturbance. These disturbance "signatures" would otherwise be obliterated by relatively vigorous transport processes or deposition rates.

The GeoSea report suggested that prop wash may aid in maintaining erosion and transport of fine-grained sediments inside the slips since the sediment distributions showed only slight net accretion. The report further suggested that sediment transport is directed both into and out of the slips.

2.4   SURFACE SEDIMENT CHEMISTRY

Contaminants of concern in the surface sediments within the study area have been identified in previous surveys. In particular, the surveys identified mercury and silver, PAHs, benzyl alcohol, butyl benzyl phthalate, and phenol and benzoic acid (EPA, 1988; Metro, 1988; Metro, 1989; Ecology, 1994; Hart Crowser, 1994). Materials in the sediment traps showed cleanup screening level (CSL) exceedences for PAHs, phthalates, dibenzofuran, 1,4-dichlorobenzene, and mercury (Norton and Michelsen, 1995). Figure 13 shows the surface chemistry (top 2 cm and top 10 cm samples) station locations and data sources that were used in describing the surficial sediment chemistry in the study area.

Contours for selected metals (mercury, silver, lead, and zinc) and organic contaminants (PAHs) show values above the SQS and CSL within the study area (Figures 14 - 19). These maps show CSL exceedences for silver, zinc, and lead in the slips between Piers 46/48, and for lead and silver between Piers 48/52. Mercury is above the CSL throughout the study area except for the capped areas, seaward of Pier 48, and a relatively small area south of the Pier 53-55 cap. LPAHs were above the CSL between the Seattle Ferry Terminal and the Pier 53-55 cap. HPAHs were above the CSL off of Pier 53, and at an earlier TPPS station located offshore of the Aquarium.

On the Pier 53-55 cap, exceedences of SQS for LPAH and HPAH occur both on the southeast and northeast corner. The southeast corner exceedence appears related to higher values south of

the cap (Romberg et al., 1995). However, the northeast corner SQS exceedence may be an artifact of the older Metro 1989 station sample located on the edge of the cap (LTDG01 on Figure 13) collected prior to capping.

These contour maps also show data gaps in the surface sediment chemistry database for the Seattle Waterfront. A discussion of data gaps is presented in Section 4.3 of this report. A discussion of the subsurface chemistry data from the three sediment cores collected during the project can be found in the Field Investigation Report (Norton and Michelsen, 1995).

2.5   SEDIMENT TRAP CHEMISTRY

2.5.1   Sediment Trap Chemistry Trends

The Volume I: Field Investigation Report (Norton and Michelsen, 1995) presents results of the sediment traps and sediment cores, and identifies chemicals of potential interest due either to the exceedence of Washington Sediment Management Standards, or their prevalence in the data at elevated concentrations. Pairs of surface (3 feet below the sea surface) and bottom (3 feet above the bottom) traps were located at stations EB1 and EB6. All other traps were located in the bottom configuration. For locations of the sediment trap stations see Figure 20.

This section presents trends and patterns in the sediment trap data, and compares them with historical surface sediment data from the Seattle Waterfront area. This evaluation is incomplete because of the lack of recent surface sediment chemistry data in some areas, and will evolve as additional data become available through further investigations and remediation along the waterfront.

Particulate concentration trends were evaluated for a number of metals and PAHs from the sediment traps. Figures 21 and 22 show the trends in concentration for selected metals (mercury, silver, lead, zinc) and PAHs, respectively, from sediment trap particulates. Zinc concentrations were scaled for clarity.

In general, metal concentrations were higher at station EB2 (mercury, silver, lead, zinc), EB5 (silver, lead, zinc), and EB8 (silver, lead) and the PAHs higher at stations EB2 through EB7. This trend correlates fairly well with the surface sediment concentrations in the vicinity of the sediment traps (see Figures 14 - 19). For example, the geometric mean mercury concentration in surface sediments in the vicinity of station EB8 equaling 1.2 mg/kg was nearly identical to average in the trap of 1.2±0.1 mg/kg (± one standard deviation). Throughout the study area, mercury concentrations exceed the CSL in all of the near-bottom traps.

Average total PAH concentration on particulates were highest in trap EB4 (300 mg/kg), located underneath Pier 56, followed by EB5 (212 mg/kg), EB3 (185 mg/kg), EB2 (175 mg/kg), and EB7 (160 mg/kg). South of the ferry terminal, traps at stations EB8 and EB9 had much lower PAH concentrations (44 and 39 mg/kg, respectively) than those further north, with the exception

of the surface trap at station EB1 (63 mg/kg).

The HPAH particulate concentration is always higher than the LPAH concentration, following a similar trend in the surface sediments. To compare the sediment trap PAH data with the existing surface sediment data, HPAH:LPAH ratios offer additional information for possibly identifying recent versus older sediments and in identifying potential sources. This is because more recent sediments should generally have a relatively larger percentage of the more mobile and biodegradable LPAH than the older sediments. Therefore, assuming a similar source of contamination, the older sediments should have a higher HPAH:LPAH ratio than the younger sediments. This evaluation can be confounded by the presence of a variety of past or present dissimilar sources. For example, petroleum contaminated subsurface sediments unearthed during demolition of the wing wall near Colman Dock had an unusually high LPAH content (KCDMS, 1994a).

Average HAPH:LPAH ratios were computed from the four quarters of sediment trap data at each station (Table 2-4). All stations had fairly constant HPAH:LPAH ratios over the study period with coefficient of variation of 17% or less except for the EB6 surface station (EB6-S) which had a coefficient of 57% due to a high HPAH:LPAH ratio in the second quarter (3.5) which fell significantly to average 1.4 in the third and fourth quarter. Stations EB1-S, EB2, and EB6-S did not have data for the first quarter.

Average HPAH:LPAH ratios in the sediment traps were statistically compared using paired t-tests to investigate significant differences (see Table 2-5). Station EB6-S, due to missing data and a high coefficient of variation, did not have a significantly different ($p < 0.05$) HPAH:LPAH ratio from any of the other stations, and is not included in Table 2-5. The other statistical comparison showed that the average from the EB1-S surface trap (3.0) and EB1-B bottom trap (2.8) were significantly higher than the other stations, but similar to each other. The trap at station EB4 had the lowest average ratio (1.2) and was significantly lower than all other traps. The traps south of Pier 54 at EB7 (2.0), EB8 (2.3) and EB9 (2.0) were not significantly different from each other, while EB8 and EB9 were significantly different from all other traps north of Pier 54. The HPAH:LPAH ratio in the trap at station EB7 was not significantly different from ratios in traps at stations EB2 and EB5.

The statistical analysis of the HPAH:LPAH ratios suggests that there are three significantly different source area signatures for particulate PAHs found in the traps: 1) north of Pier 59 the source of particulate PAH in the station EB1-S and EB1-B traps is relatively low in LPAH; 2) south of Pier 54 the source is somewhat higher in LPAH; and 3) between Piers 54 and 57 with a higher LPAH signature. The area near the end of Pier 56 and the outer Pier 56/57 slip has an even higher LPAH signature and may be the main source for the lower HPAH:LPAH ratios in the region between Piers 54 and 57.

Table 2-4 Sediment Trap HPAH:LPAH Ratios

Sediment Trap Station	Average HPAH:LPAH	Standard Deviation HPAH:LPAH	Coeff. of Variation (%)
EB1-S	3.0	0.4	13
EB1-B	2.8	0.1	3
EB2	1.8	0.2	10
EB3	1.4	0.1	5
EB4	1.2	0.2	17
EB5	1.6	0.2	14
EB6-S	2.1	1.2	57
EB6-B	1.6	0.1	6
EB7	2.0	0.3	14
EB8	2.3	0.4	15
EB9	2.0	0.1	7

HPAH:LPAH ratios were also calculated from the surface sediment data shown in Figures 18 and 19 and were found to average 4±2 (± one standard deviation) in the study area. This ratio was significantly higher than the ratio in particulates from the sediment traps at stations EB3 and EB4 ($p<0.05$) and EB5 and EB6-B ($p<0.20$). This suggests that there is an additional source of LPAH, other than resuspended surface sediments, being deposited in the traps.

The storm drains and CSOs can be ruled out as an additional source of low LPAH since their average HPAH:LPAH ratio is 3.0±1.9 in Elliott Bay (EPA, 1988). Other potential low-ratio sources include minor fuel and lubricating oil spills and leaks, potential seeps of petroleum contaminated groundwater, and possible leaching from creosote-treated pilings. The fact that the trap with the lowest ratio at station EB4 is located under Pier 56 with creosote-treated pilings, and adjacent to the tour boats dock, suggests that at least two of the possible sources may be involved.

A study of right-of-way ditches in British Columbia containing creosote-treated utility poles showed that total PAH concentrations decrease, with increasing HPAH:LPAH ratios, downstream of the utility poles (Wan, 1994). The HPAH:LPAH ratio in the ditches ranged from 0.2 near to poles to 0.6 downstream. This suggests that creosote-treated pilings could act as a low LPAH source.

Table 2-5 Results of Paired t-tests for Significant Difference in Sediment Trap HPAH:LPAH Ratios

Station	EB1-S	EB1-B	EB2	EB3	EB4	EB5	EB6-B	EB7	EB8
EB1-S									
EB1-B	-								
EB2	+	+							
EB3	+	+	+						
EB4	+	+	+	+					
EB5	+	+	-	-	+				
EB6-B	+	+	-	+	+	-			
EB7	+	+	-	+	+	-	+		
EB8	+	+*	+*	+	+	+	+	-	
EB9	+	+	+*	+	+	+	+	-	-

+ Significant difference (p < 0.05)
- Difference not significant (p < 0.05)
* p < 0.09

Silver and mercury appear reasonably correlated in the northern-most traps (EB1-S, EB1-B, EB2). This correlation extends to both the lowest and highest concentrations. The highest concentrations of silver and mercury occurred at Miner's Landing (EB2). Further south, these two metals do not appear as strongly related, although they do tend to trend together seasonally.

Lead and zinc, traditionally associated with urban stormwater sources, rarely exceeded the CSLs. They trend together very strongly, but with slightly different ratios. Throughout the seasons, zinc is approximately 3 times the lead concentration in the surface traps, but only 2 times the lead concentration in the bottom traps.

Lastly, in the southern most traps at stations EB8 and EB9, the zinc concentration was less than twice the concentration of lead, but both still trended together. Concentration of 1,4-dichlorbenzene above the CSL was detected once in the traps at station EB8 and EB9, suggesting CSO related contamination.

2.5.2 Comparisons Between Surface and Bottom Traps

Surface traps three feet below the sea surface were positioned at stations EB1 and EB6. Figure 23 presents bar charts of the chemical results for the traps at EB1 and EB6. Concentrations are scaled as in the previous figures, where the darkest bars represent the results in the first quarter (October - December 1993), while the lightest bars represent the results for the fourth quarter (July - September 1994).

Given the limited data with two surface traps, the following observations should be considered tentative:

1)   PAHs, mercury, and silver are generally higher in the bottom traps. The seasonal pattern in surface sediment traps is not reliably reflected in the bottom trap at the same location. This is especially noticeable at station EB1.

2)   Arsenic, chromium, iron, and zinc are higher in surface traps in the fall quarter (October through December 1993), but decrease to below the bottom sediment concentrations in later quarters. The surface trap at station EB1 (EB1-S) during the fall quarter had the highest measured particulate concentrations for arsenic (41 mg/kg), zinc (390 mg/kg), and iron (41,000 mg/kg) of any bottom or surface trap during the survey.

3)   Pentachlorophenol was detected both in the surface and bottom traps at EB1; it was not detected in either the surface or bottom trap at EB6. The PAHs in the surface traps were consistently lower than the bottom trap at EB1 and at EB6 until the fourth quarter when the PAHs were slightly higher in the surface trap at EB6.

The high particulate arsenic, zinc, and iron concentrations in the fall quarter EB1-S sample are believed to be from the Duwamish River plume during a high flow event during early December, 1993. Meteorological data from the Colman Dock shows at least two periods of fairly strong winds from the north during this flow event. These winds tend to constrain the plume over the study area, increasing the likelihood of capturing the particulates from the plume in the surface traps.

The Duwamish River particulate concentrations for iron, zinc, arsenic, and chromium measured by various studies average 57,000±33,000, 340±190, 38±9, and 110±18 mg/kg, respectively (Romberg et al., 1984; Curl et al, 1987; Riley et al, 1980). Thirteen of the sixteen samples reported were collected south of Harbor Island. None of these concentrations are above CSLs (note that there is no CSL for iron). These concentrations are similar to the concentrations measured in the surface trap at EB1, supporting the hypothesis of the Duwamish River plume being the source of these metals during the fall of 1993.

Summarizing the sediment trap observations for this section:

1) The co-occurrence of particulate mercury and silver concentrations in the sediment traps and surrounding sediments suggests that the majority of contamination from mercury and silver found in the bottom traps is derived from localized resuspension.

2) The HPAH:LPAH ratio data from the sediment traps suggest that there is an additional source of low LPAH besides the surrounding surface sediments, CSOs, and SDs. The source appears to be centered near the end of Pier 56 and the Pier 56/57 slip. Potential low-ratio sources include minor fuel and lubricating oil spills and leaks, potential seeps of petroleum contaminated groundwater, and possible leaching from creosote-treated pilings. The historical surface sediment data in the study area indicates that, while these sources may contribute to elevations in LPAH concentrations in the sediments, they are probably not accumulating above CSL levels.

3) The HPAH:LPAH ratio data indicates that there are three significantly different source area signatures for particulate PAHs found in the traps: 1) north of Pier 59; 2) south of Pier 54; and 3) between Piers 54 and 57. This correlates with the general near-bottom circulation pattern in the study area which shows that interior of the study area north of the Pier 52 and south of Pier 59 is hydrodynamically isolated (see Section 2.7).

4) The Duwamish River plume may be a source to the study area for a few metals such as iron, zinc, arsenic, and chromium during the high runoff periods, and when the winds drive the river plume over the study area. None of these metal concentrations are above CSLs.

2.6   PB-210 CORES AND SEDIMENT ACCUMULATION RATES

2.6.1   Pb-210 Cores and Net Sedimentation Accumulation Rates

Three cores (C1, C2, C3) were collected to determine net sediment accumulation rate (Figure 24) via Pb-210 radiometric dating (Krishnaswami et al., 1971; Koide et al., 1973). In addition, some metals and PCBs were analyzed in the core sections along with radioactive Cs-137. Methods and data associated with the cores are reported in Norton and Michelsen (1995).

Dating the cores was accomplished by first correcting the measured core section depth for core shortening (Blomqvist, 1985). Then the logarithm of the Pb-210 activity was plotted as function of corrected depth for each core (Figures 25-27) and inspected to determine the presence and depth of the bioturbated surface mixed layer. The most significant observation from these cores is that, within the resolution of the core sectioning, there does not appear to be a mixed surface layer indicative of active burrowing of benthic infauna as has been observed in most areas of Puget Sound. Consequently, the net sediment accumulation rate was determined using a constant accumulation model which considers only compaction and decay neglecting biological mixing (Christensen, 1982). This model uses the change in measured sediment bulk density with depth (compaction) to determine the net accumulation rate from the measured Pb-210 activity values.

The only other inputs to the model are the surface Pb-210 activity and the constant activity with depth supported by *in situ* Ra-226 decay. The measured bulk density as a function of depth is fitted to an exponential equation and used in the solution for the accumulation rate from a best statistical fit to the Pb-210 activity as a function of depth.

The results of the accumulation model are shown in Figures 25 through 27 where S_0 is the constant surface activity, S_∞ is the activity supported by *in situ* Ra-226 decay at depth, and r ($g/cm^2/yr$) is the resulting net accumulation rate determined from the model.

At the seabed for all three cores, the fit of the predicted Pb-210 activity to the measured values appears reasonable. However, closer inspection of core C2 reveals that the bulk density does not increase smoothly with depth as would be expected for a sediment core with constant accumulation. In the upper 50 cm there is a significant change in bulk density suggesting either a disturbed core, or that the sedimentation rate has not remained constant.

There are 4 major assumptions used in Pb-210 dating for establishing sedimentation rates: 1) the flux of excess Pb-210 to the seabed is constant; 2) the sedimentation rate is constant over time; 3) post-depositional migration of the Pb-210 does not occur in the sediment; and 4) the activity of Pb-210 from *in situ* Ra-226 decay is independent of depth. If the second assumption was incorrect the model for core C2 would not be valid.

The assumptions were checked by comparing the rates suggested for the whole core by the constant accumulation model, with rates between cores sections based on the Pb-210 age of each section. The lack of an apparent surface mixed layer allows this type of a comparison to be made. The following calculations were performed. The first order rate equation for radioactive decay may be expressed as:

$$A(t) = A_0\, e^{-\lambda t} + A_\infty$$

where A(t) is the total activity as a function of time, A_0 is the activity at the surface, A_∞ is the constant supported activity at depth, and λ is the decay constant for Pb-210 ($0.0311\ yr^{-1}$). Rearranging to solve for time, the time elapsed since deposition in years is:

$$t(years) = [-\ln\{(A(t) - A_\infty) / A_0\}] / \lambda$$

This analysis showed that in the Pier 56/57 slip, the accumulation rate in core C2 has increased, beginning in about the early 1960's, from 0.1 $g/cm^2/yr$ prior to the early 1960s to about 0.3 $g/cm^2/yr$ through the mid 1980's when it increased to its present value of about 0.7 $g/cm^2/yr$. The present rate is close to the gross accumulation rate in the two sediment traps in the slip at EB2 of 0.74 ± 0.32 $g/cm^2/yr$, 0.87±0.29 $g/cm^2/yr$ at EB3, and 0.91±0.31 $g/cm^2/yr$ at EB4 underneath Pier 56. The rates prior to 1962 in core C2 were fairly constant at about 0.1 $g/cm^2/yr$ and equaling the accumulation model results assuming constant accumulation. The higher recent accumulation rate for core C2 is consistent with the surface sediment grain size data, which show

that this is poorly sorted, fine grained area, which indicates net deposition.

Similar checks were performed for cores C1 and C3 located in the Pier 54/55 and Pier 48/52 slip, respectively. Core C1 showed some scatter in the individual accumulation rates and possibly somewhat less recent accumulation since 1983 than in the past with an average rate of 0.19 g/cm²/yr, similar to the model calculated value of 0.25 g/cm²/yr. C1 is in a slightly coarser, better sorted area than C2, possibly indicating a transition toward a less depositional and more erosional environment. The trap in the Pier 54/55 slip at EB5 yielded a gross accumulation rate of 0.79±0.23 g/cm²/yr. Core C3 had individual accumulation rates similar to the model calculated value of 0.1 g/cm²/yr in the upper sediments.

2.6.2 Gross Sediment Accumulation Rate

Gross sediment accumulation rates are determined from the mass accumulation rates in the sediment traps. These rates were evaluated with respect to seasonal effects, trends along the waterfront, and trends between surface and bottom traps.

2.6.2.1 Bottom Trap Seasonal Trends

Figure 28 shows quarterly mass accumulation rates for the bottom sediment traps. A dramatic increase in the accumulation rates in the spring and summer quarters (April 1994 through September 1994) compared with the fall and winter quarters (October 1993 through March 1994) for most stations, except EB7 and EB1 and EB9 located at the north and south ends of the study area. All other traps show an approximately 100% greater accumulation rates in the spring and summer than in fall and winter. EB1 was unchanged, whereas EB7 and EB9 showed only a 30% increase in accumulation rate in the spring and summer. Note that spring and summer are the times when the waterfront experiences seasonal increases in vessel traffic from the tour boats and the Royal Victorian auto ferry to Victoria, B.C. (see Section 3.3).

2.6.2.2 Bottom Trap Waterfront Trends

Not only do accumulation rates in the bottom traps differ seasonally, but they also vary along the waterfront. Patterns in the mass accumulation rate along the waterfront are similar between the spring and summer and the fall and winter, with the spring and summer rates higher as discussed in the previous section. The lowest rates occurred at the north (EB9) and south (EB1) ends of the study area. The largest accumulation rates are at EB8, the slip between Piers 48 and 52 where the Royal Victorian auto ferry to Victoria and Seattle passenger ferries berth. The second highest rates occurred at EB2 and EB3 located in the slip between Piers 56/57, at EB4 under Pier 56, and at EB5 in the slip between Piers 54/55. A number of Tour boats operate from Piers 55/56 with the *Sightseer* being berthed in the slip between Piers 56/57.

The lowest mass accumulation rates within the study area occur at EB6 located at the end of Pier 54, and at EB7 located south of the fire boat dock. In general, it appears that accumulation rates

are higher in areas associated with higher levels of vessel traffic, suggesting that the increase in accumulation rates is caused by an increase in the resuspension rate due to vessel traffic. The most probable mechanism is prop wash, as discussed in Section 3.2.

2.6.2.3 Bottom Versus Surface Trap Trends

Figure 29 shows the mass accumulation rate by quarter for the two pairs of surface and bottom traps located at EB1 and EB6. The trend at EB6, located at the end of Pier 56, shows that the accumulation in the surface trap is consistently lower than in the bottom traps during the fall and winter, with both the surface and bottom traps increasing in spring and summer to nearly equal rates. The increase is greatest in the EB6 surface traps equaling about a factor of 5. In fact, the surface and bottom traps at EB6 and the surface trap at EB1 each show nearly equal accumulation rates in spring and summer. This suggests that the higher resuspension and associated higher accumulation rates in the spring and summer due to increased vessel traffic observed in the bottom traps, also affects the surface traps.

The surface traps at EB1 (Figure 29) show much higher accumulations in the fall and winter than the surface trap at EB6, suggesting that EB1 receives more input from the Duwamish River plume than does EB6.

The bottom trap at EB1 does not seem to follow any trend associated with the other traps. This may be explained considering the net bottom currents (see Figure 31), and the fact that there is essentially no vessel traffic in this area. Without local vessel traffic there would be no local resuspension, and the resuspended sediments created in the heavy vessel traffic areas south of Pier 58 are transported to the south, and therefore, would not impact the bottom trap at EB1.

2.6.2.4 Accumulation Rates and Total Organic Carbon

Because the highest mass accumulation rates occurred during spring and summer along with the highest TOC levels, the question arose as to what fraction of the increase in mass accumulation rate measured in the traps was associated with increased water column productivity. An example calculation to estimate that fraction uses the data in Figure 30, which shows the TOC for the surface and bottom traps at EB1 and EB6 and the accumulation data shown in Figure 29. Figure 29 showed that the mass accumulation rate at EB6-S increased from approximately 0.2 g/cm^2/yr in the winter (January-March) to an average of 0.7 g/cm^2/yr in the spring summer, while the total organic carbon increased from 0.008 g C/cm^2/yr to 0.06 g C/cm^2/yr (obtained from multiplying the mass accumulation rate by the percent of TOC). If all of the organic carbon increase (0.052 g C/cm^2/yr) is assumed to be from diatom production, and the organic carbon is multiplied by 3.7 which incorporates the approximate ratio of dry particulate organic matter to organic carbon of 2:1 (Millero and Sohn, 1991) and the silicate to organic carbon ratio in diatoms of 1.7:1 (Parsons and Takahashi, 1972), the maximum increase in mass accumulation rate due to diatoms in EB6-S is 0.19 g/cm^2/yr or about 40% of the observed total increase. This will be a maximum estimate of diatom influence since not all of the organic carbon increase is likely to be due to primary

productivity, nor is all of the primary production from diatoms.

The same analysis was applied to the other sediment trap data using the difference between winter, which consistently had the lowest mass accumulation and TOC levels, and the average of spring and summer which consistently had the highest accumulation and TOC levels. The results of that analysis showed that most other traps had estimated maximum diatom influence on mass accumulation rates of 50% or less with EB8 at less than 20%. Two traps (EB7 and EB1-S) showed maximum diatom influence of nearly 80%. Since these are maximum estimates, the data indicate that with the possible exception of stations EB7 and EB1-S, the majority of the increase in mass accumulation rates measured in the sediment traps during spring and summer are due to increased inorganic particulate accumulation and not from increased biological productivity in the water column.

2.6.3 Comparison of Gross and Net Sediment Accumulation Rates

Table 2-6 presents a comparison of the gross sediment accumulation rates from the surface traps and net sedimentation from the Pb-210 cores.

Table 2-6 Comparisons of Gross and Net Sediment Accumulation Rates

Location	Gross†	Net†	Resuspension†	Percent Resuspended
Pier 56/57 Slip	0.81±0.43‡	0.3-0.7*	0.1-0.5	10-60
Pier 54/55 Slip	0.79±0.23	0.3	0.5	60
Pier 48/52 Slip	1.2±0.53	0.1	1.1	90

* Recent net sedimentation only; ± one standard deviation.
† Mass accumulation rates in g/cm²/yr.
‡ Average from sediment traps at EB2 and EB3.

Table 2-6 shows that the gross accumulation rates are similar in the slips between Piers 56/57 and 54/55, and higher at the Pier 48/52 slip. The data also show a trend in net accumulation and resuspension between the slips. The Pier 56/57 slip is relatively narrow without significant vessel traffic and shows the smallest percent resuspension, even though the gross accumulation is similar to the other slips. The slip between Piers 48/52 is the most open area, has the most vessel traffic, the highest gross sedimentation, and highest percentage of resuspension. These trends also show that the highest net accumulation rates occur in the narrower slips and under piers which are somewhat removed from the major vessel traffic.

The difference in sampling intervals for the two meters is an important point for this study because currents from short-term physical processes which may affect resuspension, such as vessel prop wash, will generally occur over shorter time intervals than the 15-minute sampling interval. At the beginning of this study it was not expected that currents generated by vessels would significantly influence flow and resuspension. Therefore, the sampling intervals were set to measure tidal and wind driven currents, not the more rapid short-term pulses from vessels. In the case of the Aanderra meter, the intervals chosen effectively average over these short-term events. In the case of the S4 meter, short-term events may occur while the instrument is not recording.

2.7.3 Current Meter Deployments

Figure 4 shows the locations where the Aanderra and S4 current meters were deployed during the surveys. During the first quarter, five Aanderra current meters were deployed one meter above the sea floor at Sites EB1, EB3, EB6, EB8 and EB9. To identify current speed and direction in the surface layer influenced by the Duwamish River plume, an additional Aanderra current meter was deployed at two meters depth (7 feet) below the surface at Site EB6. As data returned from the field and were analyzed, the net current vectors were displayed and inspected to reveal gaps in areal coverage. As a result of this inspection, some Aanderra sites were moved slightly from their long-term positions.

The following paragraphs explain the rationales for the current meter locations and repositioning, proceeding in order of the site numbers between EB1 and EB16. It should be noted that gaps in the current meter site numbering scheme occurred because some sites had sediment traps and not current meters.

EB1 and EB1A. During the first two quarters Aanderra current meters were deployed at Site EB1 immediately north of Pier 59. Inspection of those records suggested that the currents were influenced by flow around Pier 59. Two quarters of data at EB1 were deemed adequate, with the result that the Aanderra current meter site was permanently moved slightly offshore to Site EB1A to provide better measurements for the study area's northern boundary. Two additional quarters of Aanderra records were obtained, supplemented by an S4 current meter record.

EB2. This site was sampled with an S4 current meter because of its proximity to the University CSO, and because it was at the head of the slip between Piers 56 and 57 and eastward of Site EB3. The intent was to compare records at the mouth and head of the pier slip.

EB3. Four quarterly Aanderra current meter records were obtained at the mouth of the slip between Piers 56 and 57.

EB4. To document the effects of piers on flow along the waterfront, an S4 current meter record was obtained under Pier 56.

2.7   CURRENT METER DATA

2.7.1   Introduction

As part of this study, forty current meter records were obtained to document the near-bottom water movement along the Seattle Waterfront. The records were collected on a quarterly basis over a one year period from October 1993 through October 1994. The records were obtained at 16 locations using two types of current meters: mechanical meters manufactured by Aanderra®, Inc. (Model RCM-4), referred to as Aanderra current meters; and electromagnetic current meters manufactured by InterOcean®, Inc. (Model S4), referred to as S4 current meters (see Figure 4 for locations).

2.7.2   Current Meters

Two types of current meters were utilized because of different minimum current speed thresholds. The Aanderra current meters sense current speed using a revolving rotor known as a Savonius rotor, and a vane which swings with current direction. Because of the mechanical nature of the sensors, a relatively high current speed of 2.5 cm/s is required before the rotor begins to rotate. The S4 current meters sense current speed and direction by measuring the distortion of an electromagnetic field surrounding the spherical meter. Because the S4 current meter has no moving parts, its threshold current speed equals approximately one cm/s.

In the first quarter, only the Aanderra current meters were used in the field investigations. After the first quarter of results were analyzed, it was discovered that most of the time current speeds were below the Aanderra's threshold speed. Two S4 current meters were rented from the University of Delaware for the last eight months of the field deployments. Each month the two S4 meters were placed in new locations to provide greater spatial resolution of the near-bottom current patterns. The Aanderra current meters were deployed at the same locations to provide temporal continuity during the year of records, whereas the roving S4 current meters provided necessary spatial resolution.

The Aanderra and S4 current meters differ not only in terms of their threshold speeds and method of sensing currents, but also in the intervals of time over which the current speeds were sampled. The Aanderra current meters recorded the number of revolutions of the rotor during 15-minute intervals, thus providing an average speed each quarter hour. Current direction was recorded as the last position of the vane at the end of the 15-minute interval. Thus the speed is the average over 15 minutes, whereas the direction is instantaneous every 15 minutes.

The S4 current meters sampled with a different strategy. These current meters sense the velocity components at half-second intervals. As deployed in this project, the S4 current meters were set to record a vector average of the half-second samples during the last minute of each 15-minute interval. Thus the S4 records for this project consist of one-minute averages at 15-minute intervals.

- 35 -

2.7.4 Results

2.7.4.1 Net Currents

The net current speed and direction was obtained from each current meter by adding the speed and directional vectors obtained from each 15-minute recording over the entire current meter record. Net current vectors from all near-bottom current meter deployments are shown in Figure 31, and for S4 current meters in Figure 32. Comparison of the net current patterns from these figures indicates that net vectors obtained by Aanderra and S4 current meters yielded nearly the same spatial pattern. Therefore they were combined to develop a conceptual near-bottom current model for the Seattle Waterfront. Currents were also measured at approximately seven feet below the water surface at one location, Site EB6.

Net Near-Bottom Currents

Based on the near-bottom net currents (Figure 31), the waterfront appears to be comprised of a number of distinct flow patterns. The northernmost area is characterized by the current meter records in the vicinity of Pier 59 (EB1) where the flow outside of the pier face is northward. There may be an inshore eddy as indicated by the southward vectors at EB1A.

In the vicinity of old Pier 58 (now the Waterfront Park) the near-bottom flow changes from northward to southward. This indicates that the current is diverging away from the Pier 58 area. The record at EB10, just south of Pier 59, indicates a net vector directed to the west, which is consistent with a divergent current in the vicinity of Pier 58. However, discussions with divers suggests that numerous obstructions lie in the vicinity of Pier 59. Therefore, the current meter at EB10 may have been influenced by an obstruction.

One hypothesis which could explain a divergent current in the vicinity of Pier 58 would be the result of a hydrostatic head created by the "pile-up" of the Duwamish River plume in this area. The waterfront bulkhead turns toward the west at Pier 58, so that there could conceivably be a pile-up of surface water from the plume as it encounters this impediment. This pile-up of surface water would create a hydrostatic head which produce a divergent subsurface current.

From Pier 57 southward to Pier 54 the vectors along the pier faces point southeastward following the pier faces. The southward flow terminates near Pier 52, the Seattle ferry terminal. The two records obtained at the ferry terminal point westward. Finally, south of the ferry terminal the vectors along the dock faces point northward. In the area between Pier 52 and Pier 48, the vectors are directed southward, suggesting an eddy south of the ferry terminal.

Net Near-Surface Currents

In the study area only one meter was placed in the near-surface flow (EB6). This meter was placed seven feet below the surface so as not to interfere with vessel traffic. The net current from

EB6. An Aanderra current meter was deployed at this site during each of the four quarters of the measurement year. After inspection of the data one record was deleted because the rotor was fouled by barnacles. To provide additional confirmation of flow characteristics a little farther offshore, one of the Aanderra deployments was made offshore of the other three deployments. For additional confirmation, one deployment of an S4 current meter was made near the inshore locations of the Aanderra current meters. In addition, an Aanderra current meter was suspended near the sea surface to document the surface flow associated with the Duwamish River plume.

EB8. This site was chosen because it lay in the middle of the open area between Pier 48 and the ferry terminal. Aanderra current meters were deployed at this site throughout the year, supplemented by two S4 current meter records. One of the Aanderra records was deleted because the rotor became fouled.

EB9. This site was chosen to document the flow along the face of a pier at the southern end of the study area. From field inspections it was discovered that this location often lay under barges berthed at Pier 48. Aanderra current meters were deployed through the year at this site, where one of these records was deleted because the vane fouled. An S4 current meter was deployed in the vicinity of this site to confirm the Aanderra current records.

EB10. This site was chosen to document the flow in the relatively open area between Piers 57 and 59. Concerns arose that the flow in this area might be influenced by underwater remnants of Pier 58, which was previously removed. Interviews with divers from the Aquarium familiar with the area indicated underwater obstructions may have influenced this site.

EB11. This site was sampled with an S4 current meter to document the flow under Pier 57.

EB12. An S4 current meter was deployed near the wing wall structure located at the north side of the dual loading ramps at the ferry terminal. This site was located approximately 20 feet from the structure.

EB13. An S4 current meter was placed under the southern end of the ferry terminal in order to fill a data gap in the areal coverage and document the effects of ferries.

EB14. An S4 current meter was placed farther offshore of site EB9 and in deeper water to document flow farther offshore.

EB15. An S4 current meter was placed under Pier 48, but the record was not recovered because the meter's battery compartment flooded.

EB16. An S4 current meter was placed along the face of Pier 56 to provide added spatial resolution in the northern portion of the study area.

this location was generally toward the west. Given the complexity of the currents near the bottom (see Figure 31), it is difficult to infer an overall pattern of flow near the surface from a single current meter. There is only one other previous surface current meter record along the waterfront. It is a short record over three tidal days in 1946, and was located two meters (6.5 feet) below the surface and about 2,000 west of Pier 66 (Cox et al, 1984). The net current at this location was toward the northwest, parallel to the shoreline with a net speed of about four cm/sec.

2.7.5 Conceptual Model of Circulation along the Seattle Waterfront

2.7.5.1 Net Near-Bottom Currents

Though currents are variable at each location, the net vectors do identify an overall pattern along the piers in most of the study area as presented in Figure 33. Between Piers 48 and 57 the currents in the vicinity of the pier faces converge at the ferry terminal and then head offshore. This suggests that near-bottom water is being driven offshore at the ferry terminal, and is replaced with water drawn into the area from the north and south. It appears that water is drawn from as far north as Pier 57, and at least as far south as Pier 48.

To place the scale of these currents in perspective, the associated flow or discharge was estimated. To derive a conceptual mechanism for moving water offshore at the ferry terminal, consider the volume of water driven offshore while the ferries are idling at the terminal. A single ferry at idle discharges approximately 70 cubic meters per second (Francisco, 1995). Since ferries lie idling at the terminal approximately 40% of the time according to ferry schedules, we estimate that approximately 28 cubic meters per second are discharged to the west during a typical day.

To compare the discharge associated with the ferries, consider the volume transport associated with the current meters. To obtain a volume transport it is necessary to multiply the current speeds by a representative cross sectional area through which the current is flowing. This cross sectional area is oriented perpendicular to the shore, and measures a vertical distance by a horizontal distance. Unfortunately, our data are not spaced closely enough to accurately estimate these distances. Therefore, approximate estimates were made.

The vertical distance of the cross sectional area was taken as the depth range between the bottom of the Duwamish River plume and the seafloor, or between 10 feet and the average depth of the current meters at approximately 50 feet. The horizontal distance was estimated to be on the order of 100 meters. Multiplying the average net speed of one centimeter per second by the cross sectional area yields a volume transport of 12 cubic meters per second. Since this transport is flowing both north and south towards the ferry terminal, the total transport being drawn toward the ferry terminal is approximately 24 cubic meters per second. Therefore, it appears that the transport from the ferries approximately balances that inferred from the current meters.

maximum tidal swings into and out of the study area.

Figure 38 shows an example turbidity increase on March 31, 1994 which lasted for about ten hours. It had been suggested that this event might be related to the cap construction activities at the Port of Seattle's Pier 66 marina. However, this work was completed March 14, 1994 (Hotchkiss, personal communication, 1995). This event also does not correlate with Duwamish River flow, waves, or high winds. Furthermore, this event could be related to the major transport of less turbid water into and out of the study area during maximum tidal swings, as discussed above.

The conceptual model of the near-bottom net circulation along the waterfront (Figure 33) shows two places where currents, directed along the shoreline, change direction. At Pier 52 the Seattle ferries idling at the ferry terminal transport water offshore; currents flow toward Pier 52 from the south and the north. North of the ferry terminal at Pier 58, the net current diverges, and flow changes from flowing south to flowing north, possibly influenced by the pile-up of surface water from the Duwamish River plume against the bulkhead in this area. This circulation pattern will isolate the resuspension and deposition of sediments between Piers 52 and 59 from the area south of Pier 52.

2.7.5.2 Net Near-Surface Currents

The Duwamish River plume would be expected to dominate the near-surface circulation along the Seattle Waterfront, which apparently flows counterclockwise along the eastern shoreline of Elliott Bay (Curl et al., 1987). The plume is divided at the mouth of the river into the East and West Waterways by Harbor Island (Figure 1). Flow measurements have shown that about 80 percent of the flow passes through the West Waterway (Stoner, 1972). Near-surface salinity measurements have shown that the Duwamish River plume is generally on the order of ten feet deep, varying considerably in thickness and extent with river flow, tides, and winds. The less saline water exits the Duwamish mouth during the ebb and is subsequently pinched off by more saline water entering the estuary from Duwamish Head on the flood (Curl et al., 1987). This creates patches of less saline water that flow along Elliott Bay Waterfront, influenced by the wind which has a strong effect on the location of the less saline patches relative to the shoreline (Curl et al., 1987). Most of the fresher water in these patches is confined to the upper few feet (Curl et al., 1987)

The near-surface current meter at EB6 placed at seven feet depth might be expected to lie near the interface between the deeper saline waters and the near-surface Duwamish River plume. In the transition from surface to bottom water flow, the current vectors would reverse from north to south. The net current recorded at this location generally pointed to the west, which is consistent with the meter placed in the transition zone between surface and bottom flows. Evidence for its placement in the transition zone comes from the conductivity data from the current meter and the transmissometer which was moored at the same location and depth. The transmissometer indicated that much of the time the Duwamish River plume lay above that depth, with the plume occasionally deepening as shown by depressed light transmittances. This was also seen as a lowering in the conductivity. Given these results, it appears that the current meter was placed too deep to represent the flow in the main portion of the Duwamish River plume.

2.7.6   Tidal Currents and Current Roses

The net current vectors represent the average current and do not represent the variability of the currents as measured at a single location. To show the variability, the speed and direction of the measured currents are displayed as a function of geographical directions as in a compass rose. These current representations are termed 'current roses.'

Two types of current roses were prepared for this report: percent occurrence roses which indicate the percentage of time in which the current was directed in a given direction; and mean current speed roses which show the average speed of the currents in a given direction. Percent occurrence and mean current roses were prepared for each quarter as shown in Figures 34a through 34d and 35a through 35d, respectively.

The tidal currents are best represented by the current roses. The mean current speed roses indicate that mean current speeds are rarely above 5 cm/sec (Figures 35a through 35d). The current speed roses show speeds in all directions which is indicative of the cyclic tidal currents. The percent occurrence roses show preferences for directions in and out of slips, along the piers, and away from shoals. Mean currents tend to be directed in and out of slips, and are significantly dampened under piers and in the inner portions of the slips.

2.8 SUSPENDED SEDIMENTS

To measure suspended sediments in the water column, three 25-centimeter beam transmissometers were deployed in a vertical array at the end of Pier 54 (Figure 36). The instruments were fixed at 2, 10 and 20 feet above the bottom, at a depth of 23 ft MLLW (mean-lower low water). They were installed on 12/23/93 and removed on 5/26/94. Every two weeks, the transmissometers were inspected and cleaned, and the data files were downloaded.

The growth of marine organisms on the instruments' optical surfaces caused the signal strength to decrease over time. A qualitative analysis of the data can be made by comparing sharp increases or decreases (spikes) in the percent transmittance associated with changes in turbidity. There were many such sharp increases in turbidity recorded in the transmissometer data; and only a limited attempt was made to correlate turbidity spikes with other field data.

Three examples of turbidity spikes are presented in Figures 37 and 38. The upper transmissometer with the sharp decreases in transmittance shown in Figure 37a clearly shows the effect of the relatively turbid Duwamish River plume. River flows at the time of these spikes were relatively high. Each surface turbidity maximum occurred at low tide. Since the transmissometer was moored at a fixed depth, the spikes may be interpreted as the lowering of the Duwamish River plume with the ebbing tide to the depth of the instrument.

Another kind of event was recorded by the instrument nearest the bottom. Turbidity increased sharply and repeatedly over a period of several days in Figure 37b. As currents shifted, kelp (or debris) might have been swept in and out of the light path. However, the last three events persisted about 12 hours, suggesting the time period between the maximum tidal swing from major ebb to major flood. This tidal swing would transport the majority of water, and occurs every 12.4 hours. The maximum tidal swing would also be the time of maximum tidal current, or about 5 cm/s. Assuming that less turbid bottom water exists north of Pier 58 due to the lack of resuspension from vessels, then the time required to reach the transmissometer at Pier 56 is 2.5 hours. Therefore, these events could be related to the major transport of less turbid water during

3.0 SEDIMENT RESUSPENSION AND TRANSPORT

3.1   INTRODUCTION

Sediment resuspension is a dynamic process which occurs when water and sediment interactions exceed certain limiting factors. Resuspension is dependent upon the overlying water having sufficient velocity or exerting sufficient force through turbulent shear stress to cause the individual sediment grains to begin to move. Once the sediment grains are in motion, the water must maintain sufficient turbulent velocity or capacity to keep the sediment in motion. Sediment motion is generally dependent on a combination of sediment size and water velocity. Sediment motion includes rolling along the bottom as bed load sediments, bouncing along the bottom as saltating sediments, or being swept up into the water column and carried along with the water as suspended sediments. If the water velocity or shear stress never rises above that required for motion (erosion or resuspension), the sediment will not move. If the transporting water velocity or shear stress decreases to below that required for continued sediment transport, the sediment will fall out of the water column and redeposit on the bottom. The process is complicated by spatial and temporal variations in the water velocity field, bottom sediments consisting of a mixture of various sizes, and biological activity altering the sediment structure. Sediments can also be resuspended by biological activities (bioturbation) and human activities, such as pile driving and pulling and other forms of waterfront construction. The impacts of these activities are even less understood than for water dynamics and are not discussed in this report.

The transport of sediments has been examined in theoretical treatises, in the laboratory, and in the field. Investigations into the initiation of sediment motion by water dynamics has resulted in two traditional methods for describing and reporting the results. The initiation of sediment motion, resuspension in our project, can be described as the water velocity measured 1 meter above the bottom which begins to cause sediment movement. In another approach, shear stress at the incipiency of sediment motion is calculated at the sediment-water interface.

Water velocities 1 meter off the bottom required to resuspend sediments are shown on Figure 39 as a function of the mean sediment grain size. The band separating resuspended sediment from unsuspended or static sediments is a composite of results from several investigations (Hjulstrom, 1939; Sundborg, 1967; Postma, 1967). A similar band for shear stress as a function of mean sediment grain size is shown on Figure 40. The latter figure is based on the work of Shield (1936) and modified from data presented in Miller et al. (1977) for Elliott bay sediment characteristics. The bottom velocities or shear stresses needed to initiate sediment movement and resuspension are not well defined. This is indicated by the broad band shown on both figures, as opposed to a fine, sharp line. The general trend is for the incipient motion or shear stress to increase with increasing sediment size for cohesionless sediments (sand and coarser) which is where this approach has been applied.

For fine sediments (silts and clays), which constitute more than 90% of material in the sediment traps, the incipient motion, or critical shear stress/velocity relationship is less well understood. Electrochemical and biochemical characteristics seem to have a profound influence on the initiation of sediment movement of fine materials, contributing to the uncertainty and broadness of the band in the figures. Further uncertainties contributing to the spread of the band include the difficulty of establishing the exact initiation of grain motion, and differences associated with accelerating and decelerating water velocities on resuspension and settling.

Due to the uncertainties in applying the traditional critical shear stress/velocity relationship to determine the resuspension potential for fine sediments, the recent sediment transport literature was reviewed yielding the approach described in the following section.

3.2   SEDIMENT RESUSPENSION MODEL FOR FINE SEDIMENTS

The present understanding of turbulent motion and shear stress at the sediment bed suggests that some particle movement occurs at all turbulent velocities (Lavelle and Davis, 1987). That is, there is no actual critical motion-no-motion transition or critical threshold velocity as the term has come to imply.

The impact of this understanding is more pronounced for the finer particles which are more easily moved and entrained by the turbulent flow. Therefore, the resuspension or erosion rate for silts and clays will be greater than for sands. Since smaller particles also have slower settling velocities, they tend to remain above the bed longer once they are entrained into the turbulent flow. Therefore, research has centered on measuring the erosion rate of the finer sediments as a function of the applied velocity and shear stress (e.g., Kuijper et al., 1989; Parchure and Mehta, 1985).

The measurement and dynamics of the fine-grained particle resuspension or erosion rate has also been recently studied in Puget Sound sediments by Lavelle and colleagues at NOAA's Pacific Marine Environmental Laboratory (Lavelle and Davis, 1987; Lavelle et al., 1984). They found an empirical relationship between the measured erosion rate and the bed shear stress:

$$ E = \alpha \left| \frac{\tau_b}{[(\rho_s - \rho)gD_{50}]} \right|^{\eta} $$

where: E = erosion rate (g/cm²/sec)

τ_b = bed shear stress (dynes/cm²)

η = empirical erosion rate power term (unitless)

α = empirical erosion rate coefficient (g/cm²/sec)

D_{50} = mean particle diameter (cm)

ρ_s = sediment density (g/cm^3)

ρ = water density (g/cm^3)

g = acceleration of gravity (cm/s^2)

Since the measured accumulation rates in sediment traps represent the net sum of local and non-local resuspension (i.e. erosion) and transport processes along the waterfront integrated over the deployment period, the equation above may be used to assess the potential contribution from local resuspension processes. The major assumption in applying the equation is that the measured accumulation rate from the sediment trap equals the time-averaged erosion rate.

Table 3-1 shows the measured accumulation/erosion rates from each of the traps and the bed shear stresses calculated from the above equation using the nearby grain size data for the first two quarters (October 1993 through March 1994) of the study period. Table 3-2 shows calculated bed shear stresses for the second two quarters (April '94 through September '94).

These erosion rates and calculated bed shear stresses represent the average of three calculations using erosion rate constants (α and η) measured from three fine-grained cores in Puget Sound (Lavelle and Davis, 1987). According to these authors, the rate constants represent erosion of the more consolidated sediment bed below the very thin (<1 mm) watery surfical "fluff" which was shown to be more easily eroded.

As discussed in Section 2.6, there is a significant increase in the net erosion rate in the spring and summer at all trap locations except for EB1-B, EB7 and EB9. This is reflected in the calculated shear stress as shown in Tables 3-1 and 3-2. Possible local erosional sources include prop wash, wind waves, ship wakes, and tidal currents.

Tidal currents were eliminated as a significant source of local resuspension over most of the study area. This can be shown by calculating the apparent erosion velocity required to produce the measured erosion rates. This is accomplished by applying the quadratic stress equation, based on uniform steady flow, given by the following equation.

$$\tau_b = c_D \, \rho \, U^2$$

where: τ_b = bed shear stress (dynes/cm^2)

U = velocity (cm/s)

c_D = frictional drag coefficient (unitless)

ρ = water density (g/cm^3)

Tables 3-1 and 3-2 show the apparent erosion velocity (U) calculated from the above equation using a drag coefficient of 1.6×10^{-3}, based on tidal current studies in Puget Sound (Lavelle et al., 1984). Tidal currents measured in the study area were less than 5 cm/s, which is significantly less than the apparent erosion velocities required to produce the measured erosion rates,

suggesting that tidal currents are not a significant erosional process in the study area.

Table 3-1 Calculated Net Erosion Rate, Bed Shear Stress, and Apparent Erosion Velocity from Sediment Trap Gross Accumulation Rates October 1993 - March 1994

Station	Mean Grain Size D_{50} (microns)	Gross Accumulation Rate Q1 & Q2 ($g/cm^2/yr$)	Erosion Rate ($g/cm^2/sec$)	Bed Shear Stress ($dynes/cm^2$)	Apparent Erosion Velocity (cm/s)
EB1-B	150	0.48	1.52E-08	0.22	12
EB2	90	0.47	1.49E-08	0.13	9
EB3	90	0.63	1.98E-08	0.17	10
EB4	100	0.65	2.05E-08	0.20	11
EB5	60	0.59	1.87E-08	0.11	8
EB6-B	150	0.40	1.25E-08	0.18	11
EB7	100	0.57	1.79E-08	0.18	11
EB8	90	0.73	2.30E-08	0.20	11
EB9	60	0.46	1.44E-08	0.08	7

Table 3-2 Calculated Net Erosion Rate, Bed Shear Stress and Apparent Erosion Velocity from Sediment Trap Gross Accumulation Rates April 1994 - September 1994

Station	Mean Grain Size D_{50} (microns)	Gross Accumulation Rate Q3 & Q4 ($g/cm^2/yr$)	Erosion Rate ($g/cm^2/sec$)	Bed Shear Stress ($dynes/cm^2$)	Apparent Erosion Velocity (cm/s)
EB1-B	150	0.53	1.68E-08	0.25	12
EB2	90	1.02	3.22E-08	0.28	13
EB3	90	1.11	3.52E-08	0.31	14
EB4	100	1.17	3.71E-08	0.36	15
EB5	60	0.99	3.12E-08	0.18	11
EB6-B	150	0.70	2.20E-08	0.32	14
EB7	100	0.76	2.39E-08	0.23	12
EB8	90	1.59	5.04E-08	0.45	17
EB9	60	0.58	1.84E-08	0.11	8

Therefore, waves, wakes, and prop wash remain as potential local erosional sources for the resuspended fine sediments found in the traps. These potential erosional sources are discussed in the following two sections.

3.3   PROP WASH RESUSPENSION

3.3.1   Prop Wash Literature Review

In the application of sediment resuspension models to coarser-grained sediments, it has been frequent practice to use the threshold criteria for sediment particle motion and resuspension developed for uniform steady flow in the water above the sediments. As discussed in section 3.1, Shield's threshold-type curves (see Figures 39 and 40) were empirically derived for uniform, steady flow conditions. However, prop wash is neither a steady nor a uniform flow (i.e., the cross sectional area of the flow and the discharge vary with time and distance). Instead, it is a turbulent circular jet which expands outward from the propeller within a cone angle which has been measured between about 20 to 30 degrees (Blaauw and van de Kaa, 1978; Hamill, 1988; Verhey, 1983; Fuehrer, 1987).

Under most conditions a propeller jet impacts the sediments in a fully turbulent condition at a relatively shallow angle. The turbulent flow and shallow angle of the jet increase the actual shear stress on the sediments over what would be expected in uniform, steady flow conditions. In fact, empirical studies of coarse-grained sediments impacted by propeller jets have shown that particle motion begins at lower threshold velocities than Shield's criterion would indicate (see Hamill, 1988).

Based on this literature review, it is appropriate to rely on the relevant empirical studies over purely theoretical models of coarse-grained sediment motion from propeller-generated turbulent jets. These studies provide more appropriate sediment motion criteria than Shield's curve.

3.3.2   Measured Vessel Prop Wash Currents

Recall that the S4 current meters were set to record one-minute average velocities once every 15 minutes and the Aanderra meters recorded 15-minute averages every 15 minutes. For small, fast-moving vessels, a one minute sampling burst is too long to record peak velocities. It is also unlikely that the propeller-current will flow in the same direction for an entire minute given changing rudder angles as a vessel maneuvers. For these reasons, it is likely that current measurements in this study substantially underestimate maximum short-term velocities due to prop wash.

However, in spite of these limitations, it appeared that current meter location EB14 was in a position to measure propeller currents from the passenger ferry dock, from the *Royal Victorian* Canadian auto ferry at Pier 48, and possibly from container ships or other vessels south of Pier 48.

- 47 -

From the plot of current speed as a function of time of day at EB14 (Figure 41), it is evident that the ambient tidal currents are no more than about 5 cm/sec. High velocities recorded between 12:00 and 1:00 PM coincided with the scheduled arrivals and departures of the *Royal Victorian*. The *Royal Victorian* idles at 200 RPM with ½ foot of pitch while alongside Pier 48. Model results indicate that the Royal Victorian generates fairly high bottom velocities, but the current meter was positioned too close to the stern of this vessel to capture the maximum bed velocities generated by her propellers. The meter position at EB 14 was also off-center with respect to the centerline of the Royal Victorian.

High velocities late in the evening and early in the morning shown in Figure 41 coincide, in most cases, with movements of the *Tyee* as recorded on vessel data cards at the Seattle office of the Puget Sound Vessel Traffic Service. Figure 42 shows speed and direction for the higher short-term velocity spikes seen in the current meter record from EB14. The grouping of high speeds heading from between about 240 and 270 degrees shown in Figure 42 indicates that these high-speed spikes came from the direction of the Passenger-only ferry dock. Vessel Traffic Service records also indicate that the *Tyee* was operating in the vicinity of Pier 48 when many of these high velocities were recorded.

The grouping of high speeds heading from between about 0 to 20 degrees shown in Figure 42 indicates that these high-speed spikes came from the direction of the East Waterway along Pier 46. Because the Hanjin container ships operate in this area, it was suspected that these high-speed spikes may be generated by the container ships or tugs used to guide them to berth.

3.3.3   Vessel Prop Wash Investigation Current Meter Deployments

Two independent investigations of the effects of prop wash on sediments along the Seattle Waterfront were undertaken during the study period. One investigation was conducted by the Washington State Department of Transportation (WDOT) to determine the effects of the passenger-only ferries on contaminated bottom sediments south of Pier 52 (Hartman Associates, 1995). A second study was conducted by Michael Francisco with the National Oceanic and Atmospheric Administration (NOAA) as part of his master's thesis for the University of Washington School of Marine Affairs (Francisco, 1995).

As these parallel investigations proceeded, information was shared among the three projects. As part of this cooperation, two S4 meters were deployed for two days between October 25-27 at two locations (EB8 and EB16) in an attempt to measure the effect of prop wash from passenger ferries on bottom currents. The meters were set to sample every half-second and to record 30 second averages continuously for the two days.

One of the meters was placed at the end of Pier 56 (EB16), and the other 30 meters south of the passenger ferry dock at Pier 51 (EB8-B). EB8-B was positioned specifically at the request of a consultant for WDOT to record velocities generated by the *Tyee* (Shepsis, personal communication, 1995), and was apparently placed at the base of a bathymetric slope south of the

passenger-only ferry dock (Francisco, personal communication, 1995). The S4 storage capacity allowed for continuous data recording up to approximately 2.5 days. Velocities of up to 15 cm/sec were recorded at EB16, and velocities of up to 7 cm/sec were recorded at EB8-B.

The higher velocity spikes recorded at EB14 (10-25 cm/s) discussed in the previous section, were not seen in the EB8-B record. The velocity spikes recorded at EB14 were relatively rare events, occurring in less than 1% of the total of 2,605 observations over a period of 27 days, and only in the late evening and early morning hours. It is possible that the *Tyee* or some other vessel is conducting operations in the vicinity of Pier 48 in the late night/early morning hours, which were not recorded by the EB8-B meter due to its placement at the bottom of the slope, but the apparent discrepancy between these two records remains unresolved. At the time of this writing, the WDOT study results are not yet available for review.

3.3.4 Vessel-Generated Bed Velocities on the Seattle Waterfront

In order to determine bed velocities and potential erosion rates from vessel prop wash, informal interviews of vessel operators on the Seattle Waterfront were conducted during the summer of 1994 (Francisco, 1995). The purpose of the interviews was to obtain inputs for a screening-level model of propeller-jet velocities expected as a result of vessel operations on the waterfront. The model used is that of Fuehrer et al. (1987). This model was chosen because it was one of the more recent models which took into account the fact that the propeller jet is not free and is limited by the bottom and water surface. It also included empirical coefficients for the effects of a central rudder splitting the jet, and for a jet created by a twin-screw vessel. These coefficients were based on scale model tank tests. The final form of the equation used is as follows:

$$V_{x,r} = V_0 \, A \left(\frac{x}{D_p} \right)^{-a} Exp \left[-\frac{1}{2C^2} \, \frac{r^2}{x^2} \right]$$

where: $V_{x,r}$ = propeller jet velocity at distance x behind and r below the propeller axis (cm/sec)
D_p = propeller diameter (cm)
V_0 = initial velocity behind the propeller = $n \, D_p$ where n is the propeller shaft rotation rate (rev/sec)
C = tangent of the angle at which the velocity field expands behind the jet
A = empirical coefficient related to the effect of the rudder splitting the jet
a = empirical coefficient related to surface and bottom limitations and the effect of twin propellers

The model results are shown in Table 3-3, and are discussed in greater detail by Francisco (1995).

Table 3-3 Predicted Bed-Velocities Generated by Vessels on the Seattle Waterfront

Vessel Name	Water Depth (m)	Shaft RPM	Props	Max. Bed Velocity (cm/s)	Position of Max. Bed Velocity (m)
Sightseer	9	180	1	14	62
Sightseer	9	360	1	28	62
Spirit of Seattle	9	180	2	91	95
Spirit of Seattle	12	180	2	59	131
Spirit of Seattle	9	360	2	182	95
Goodtime I, II, III	9	180	2	52	94
Goodtime I, II, III	9	360	2	104	95
Alki	9	180	2	68	94
Alki	9	360	2	135	94
Chief Seattle	9	180	3	48	96
Super-Class Ferries[1]	12	50	1	47	64
Super-Class Ferries[1]	12	90	1	87	64
Super-Class Ferries[1]	15	90	1	64	89
Tyee[2]	9	300	2	136	88
Tyee[2]	9	360	2	164	88
Tyee[2]	9	480	2	218	88
Royal Victorian[3]	15	90	2	89	153
"Container Ship"[4]	15	30	1	122	68
"Container Ship"[4]	15	50	1	195	68

[1] "Hyak", "Kaleetan", "Yakima", "Elwha"
[2] No rudder was used in modeling the Tyee
[3] The "Royal Victorian" has twin variable pitch propellers. This result was obtained by assuming twin fixed-pitch propellers operating at a shaft RPM of 100.
[4] Generic container ship with a 9 meter diameter propeller operating at slow speed.

Given the survey information and the model results, vessels on the waterfront are capable of generating bed velocities above 10 cm/sec. Assumptions used in generating the model results are listed below:

The bottom slope was assumed to equal zero. The effect of bottom slope may either increase or decrease the shear stress from the propeller jet, depending on jet direction relative to the slope of the bed. Vertical surfaces such as bulkheads or wingwalls were neglected. Vertical structures divert momentum to the bed, thereby increasing bed shear stress in their immediate vicinity. The model results are depth-dependent. The preliminary results presented here were generated by assuming an average depth in the area that each vessel operates.

The modeled bed velocities range from 14 to 218 cm/sec. This is equivalent to a bed stress of from 0.3 to 5 dynes/cm^2 using the quadratic stress equation from Section 3.2. This is a conservative estimate since it is based on uniform steady flow. These predicted prop wash bed stresses are equal to or greater than the stresses required to produce the net erosion rates measured from the sediment traps. However, since the sediment traps measure the net accumulation integrated over time, and prop wash resuspension is not a continuous process, we can not directly quantify the overall relative contribution of prop wash to the net erosion rate.

From Figure 42, it can be seen that some of the maximum measured velocities at EB14 are consistent with modeled velocities for the passenger ferry *Tyee*, given the 15-minute time averaging of the current meter, the short length of time for the prop wash, and that speeds in excess of 10 cm/sec originate from the direction of the passenger ferry dock.

3.4 WAVE AND WAKE RESUSPENSION

Waves in Elliott Bay are primarily wind- or vessel-generated (i.e. wakes). Waves may also be generated by a landslide from a perimeter bluff into the bay, an explosion in or on the bay, or a submarine landslide; but these events are considered to be too infrequent, random, and localized for further consideration.

There are no known studies of waves along the Seattle Waterfront. Therefore, waves for this study were predicted from available wind data using the bathymetry of Elliott Bay. The predictions were performed using the Automated Coastal Engineering System (ACES) of the U.S. Army Corps of Engineers - Coastal Engineering Research Center (1992).

Wind speed and direction records for Seattle from 1954 to 1989 were examined to predict wave conditions along the waterfront. Winds in Elliott Bay were measured by a wind sensor at the old Federal Building until 1964, when the sensor was moved to SeaTac airport. The one-hour peak winds from west-southwest (WSW) through northwest (NW) for the period of record are shown on Table 3-4. The highest wind speed, assumed to be a 20-year peak, was 39 miles per hour (mph). The average wind speed for the same period equaled 9 mph. A 1-year peak was assumed to be about 28 mph. The wind speeds shown in Table 3-4 resulted in the wave heights and

periods shown in Table 3-5.

Vessel generated waves were predicted using a model developed by Weggel and Sorenson (1986) as shown in Table 3-6. Wave heights and periods were simulated as a function of speed for a tugboat and the fireboat. These vessels each produce about 6 to 10 large waves.

Estimates for wave and wake resuspension potential were determined using the maximum wave-generated bottom velocities from linear wave theory (CERC, 1992). The wave characteristics were used to obtain an estimate of the maximum horizontal bottom velocity 1 meter off the bottom as a function of water depth. Maximum wave and wake-induced velocities near the bottom last only a fraction of a second under the trough and crest of the waveform, and quickly drop to zero as they pass over the seabed. These bursts of velocity resuspend sediments into the water column. Almost immediately, the coarser-grained sand and gravel settle back to the bottom. Finer grained silts and clays, may stay in suspension much longer.

If the fines are suspended high enough into the water column, they will not settle back to the bottom before the next wave arrives. This will keep the fines in suspension allowing them to be transported by the prevailing currents. Based on settling velocities for silts and clays, using the typical one-year average 1-foot wave period of 2 seconds (see Table 3-5), all silts and clays will be kept in suspension that were suspended at least 0.5 cm above the bed. This height of rise is probably common for a 1-foot wave. This suggests that the average waves along the waterfront could resuspend or keep in suspension the fine-grained sediments.

Wave- and wake-induced bottom velocities as a function of depth are shown in Figure 43. One-year average waves generate maximum bottom velocities exceeding 10 cm/sec in depths less than seven feet. In ten feet of water this maximum velocity decreases to less than 5 cm/sec. The 10 cm/sec bottom velocity is equivalent to a bottom shear stress of about 0.16 dynes/cm^2, exceeding the minimum net erosion rates calculated from the sediment traps. This suggests that one-year waves in less than seven feet of water could potentially contribute to the net resuspension measured in the nearshore sediment traps.

The one-year and 20-year maximum waves would resuspend some sediment into the water column. However the net resuspension from these events would be small, most likely occurring during fall and winter storms.

Figure 44 shows the bathymetric map for the study area with the 7-foot contour indicated as a potential resuspension zone for the 1-year average wave. These areas are located adjacent to Pier 46, from north of S. Washington St. to the fire dock, and from Pier 57 north to the Aquarium. The shoreline between Piers 54 and 57 is greater than 10 feet deep.

If waves were a major source of resuspension and accumulation measured in the traps, higher accumulation rates during the fall and winter storms would be anticipated. In fact the opposite was observed with higher rates occurring in the spring and summer. Furthermore, higher rates in

the traps adjacent to the shallow near-shore areas are expected if waves or wakes were a major source. This is not the case since traps at EB7 and EB1 are within 200 feet of shallow areas and have the some of the lowest rates regardless of season. The only possible exception is EB8 which shows somewhat higher accumulation rates in the fall and winter than the other sediment traps.

Table 3-4 Peak Monthly Winds WSW through NNW Measured at SeaTac Airport

Year	Month	Direction	Speed (mph)
1989	March	W	27
1985	July	NW	18
1973	June	W	21
1971	October	NW	26
1966	April	NNW	26
1959	June	WSW	22
1958	November	NW	39
1957	September	NNW	31
1955	May	WSW	35
1954	May	WSW	28
1954	September	WSW	24

The maximum bottom velocities caused by ship wakes are similar to those caused by waves (Figure 43). In an emergency, a fireboat traveling at 20 knots generates about a 20-year maximum wave, and a tugboat traveling at 10 knots a one-year average wave. Since these events are much less frequent than the average wave which arrives every 2 seconds, their overall contribution to the net resuspension would be minimal.

Overall, the contribution of waves and wakes to the net resuspension of the fine-grained material measured in the sediment traps is considered minor, except possibly in areas immediately adjacent the bulkhead which receive significant fine-grained input (such as near the S. King St. CSO). Wave and wake action along the waterfront is regulated vertically by the tides. As described above, the mean variation in the water level along the waterfront due to tides is 11.35 feet. With a sloping bottom in front of the Alaska Way seawall, the seaward limit of wave action will move in and out as the tide level rises and lowers.

Table 3-5 Predicted Wind-Generated Wave Characteristics
for the Seattle Waterfront

Statistic	1-Year Average	1-Year Maximum	20-Year Maximum
Wind Speed, mph	10	28	39
Wind Speed, Knots	9	24	34
Wave Height, ft	1	3	5
Wave Period, sec	2	4	5

Table 3-6 Ship-Generated Wave Characteristics

Statistic	Tugboat	Tugboat	Fireboat
Ship Speed, Knots	5	10	20
Wave Height, ft	0.5	1	5
Wave Period, sec	2	3	6

4.0 CONCEPTUAL SITE MODEL AND RECOMMENDATIONS

4.1 CONCEPTUAL SITE MODEL

The conceptual site model is built upon an understanding of the study area from the data evaluation presented in the previous sections. It ties together the observations and conclusions regarding the major processes at work along the waterfront controlling contaminated sediment source, transport and deposition.

The conceptual model for the major waterfront processes is diagramed in Figure 45. The controlled discharge in the figure represents the conclusion that outside of localized impact from the S. King St. CSO, point source discharges including the Duwamish River are not a likely source of recontamination.

The major source of resuspension represented in Figure 45 is from prop wash in vessel traffic areas, with a minor amount occurring in shallow areas next to the bulkhead due to wave action. In vessel traffic areas, there is less net accumulation and greater resuspension from prop wash. In protected areas away from vessel traffic, such as under piers and in narrow slips, there is less resuspension and greater net accumulation.

The conceptual model of the near-bottom net circulation along the waterfront (Figure 33) shows two places where currents, directed along the shoreline, change direction. At Pier 52 the Seattle ferries idling at the ferry terminal transport water offshore; currents flow toward Pier 52 from the south and the north. North of the ferry terminal at Pier 59, the net current changes from flowing south to flowing north. This circulation pattern isolates the resuspension and deposition of sediments between Piers 52 and 59 from the area south of Pier 52.

Based on the conceptual model and the sediment trap and surface sediment data, the overall conclusion is that resuspension of contaminated surface sediments from vessel prop wash is the major source of particulate contaminants above the CSL found deposited in the sediment traps, and is therefore the most likely ongoing source for recontamination in the study area. The King St. CSO may also have a localized impact on the slip between Piers 46 and 48.

4.2 RECOMMENDATIONS

The conceptual model of the Seattle Waterfront indicates that circulation patterns isolate the resuspension and deposition of sediments between Piers 52 and 59 from the area south of Pier 52. Since vessel traffic in these areas cannot be controlled, it is recommended that remediation planning should include all of the area between Piers 52 and 59 as a single cleanup area to minimize the potential for recontamination from vessel prop wash. Similarly, the area south of Pier 52 to Pier 46 should be considered as a single management unit for remediation.

Since vessel prop wash may also affect in-place remedial actions such as capping, the recommended approach was to review existing prop wash and scour models and utilize the most appropriate model or models for determining prop wash scour potential for likely in-place remedial designs. Section 6.0 discusses the models and their usefulness and limitations toward determining prop wash scour potential for in-place remedial designs.

5.0 DATA GAPS

This section focuses on identification of relevant data gaps that could be filled as part of the site characterization. The purpose of this section is to identify the areas, types of samples to be collected, and analyses to be performed in order to fill the data gaps. This discussion is not intended to direct or require a specific sampling and analysis scheme, but instead to offer suggestions to the Panel that could be useful for completing the site investigation and evaluating remedial alternatives. A detailed sampling and analysis plan is planned to be developed by the City of Seattle to complete the cleanup study.

5.1   SURFACE SEDIMENT CHEMISTRY DATA GAPS

Potential contaminants of concern in the surface sediments within the study area have been identified in previous surveys and are discussed in Section 2.4. In general, however, areas under piers and nearshore areas have not been well-characterized. Figure 46 shows the surface chemistry station locations within the study area. Three areas labeled A, B, and C are delineated and identified as surface chemistry data gap areas for purpose of discussion.

Data Gap Area A extends from the northern boundary of the site on the south side of the aquarium at Pier 59 down to and including Pier 56. There are only 4 stations in this area, two of which were collected in 1985 and are now 10-year-old data (Tetra Tech, 1988). The Ecology C2 sample was taken from the surface section of a core collected for radioisotopic (Pb-210 and Cs-137) analysis to determine net sediment accumulation rates, and was not evaluated to determine total depth of contamination. Consequently, the core section samples were analyzed for some metals and PCBs but not for PAHs or silver. Therefore, with the exception of Station UP4, the data in this area are either out of date or of limited usefulness for identifying the full suite of the potential chemicals of concern.

To delineate the northern boundary of the site, two samples could be collected from under Pier 56, three from under Pier 57, two from the slip between Piers 56 and 57 and three in the vicinity of Waterfront Park. Example locations are shown in Figure 46. Analyses would be performed for metals (arsenic, cadmium, copper, lead, mercury, silver and zinc), semivolatile organics (which would cover PAHs and the other potential organic chemicals of concern), TOC, and grain size.

Area B extends from the south side of Pier 56 to and including the fire dock. There are only four recent stations in this area (UP1, UP2, UP3, and C1) including the Ecology core C1, which does not include the potential organic chemicals of concern or silver as described above for core C2. Station UP3 under Pier 55 would be adequate to characterize this area. However, Station UP3 does not tell us about the area between the Seattle harbor tours dock at Pier 55 and the Pier 53-55 cap. Even though Station LTDG01 is located on the edge of this area, these data are now six years old. Furthermore, this sample was located next to the Pier 53-55 cap and has most likely

been influenced by the capping material placed following its collection. Data are also lacking in the slip between Piers 54 and 55 and inside of the end of the fire boat dock. Two additional stations could be located west of the Seattle harbor tours slips to confirm whether that area requires remediation. Two stations could also be located in the vicinity of the fire dock including one station under the main dock. Example locations are shown in Figure 46.

The area between areas B and C has been well characterized for metals and PAHs in recent investigations by the Washington State Department of Transportation (WSDOT) as part of the planned renovation of the Seattle Ferry Terminal, and by METRO as part of their monitoring program for the Pier 54/55 remediation cap. This area would not need further characterization.

Area C extends from the near shore area on the south side of Pier 52 out to the end of Pier 48 and the slip between Piers 46 and 48. The portion of Area C under, and just north of, Pier 48 has no recent stations. This portion of Area C could use three to four stations north of Pier 48 and two to three stations under Pier 48 to help delineate the influence of the S. Washington St. storm drain from that of the S. King St. CSO.

Of the seven existing stations in the slip between Pier 46 and 48, one is 10 years old, four are 7 years old and two are 6 years old. Two to three additional confirmation samples could be collected in the slip between 46 and 48. The slip stations would include a station near the South King St. CSO for source delineation, since this CSO was identified as a potential minor source in this study. Analyses performed in Area C would include metals (arsenic, cadmium, copper, lead, mercury, silver and zinc), semivolatile organics (which would cover PAHs and the other potential organic chemicals of concern), TOC, and grain size. Example surface sediment sampling locations are shown on Figure 46.

5.2   SEDIMENT CORES DATA GAPS

Figure 47 shows the location of recent sediment cores collected in the study area and suggested potential locations for additional cores. The Ecology cores (C1, C2, and C3) were collected in the slips by gravity corer in 1994 with penetration depths of less than 2 meters. These cores were used to collect samples for radioisotopic (Pb-210 and Cs-137) analysis along with selected metals and PCBs to determine net sediment accumulation rates, and were not taken to determine total depth of contamination. The cores collected for WDOT (designated WSF) were deeper hollow stem auger borings which penetrated to 6 meters.

The deeper WDOT cores were analyzed for PAHs and metals and showed LPAH, HPAH, and mercury exceeding the CSL down to about 3 meters (10 feet). The Ecology core C2 in the slip between Piers 56 and 57 showed the highest mercury levels of all cores at about 2 meters with concentrations over 10 mg/kg (CSL 0.58 mg/kg). However, there are no deeper cores in the areas A or B to suggest how widespread or how deep this mercury contamination extends. There are no borings in Area C.

Since dredging may be necessary as part of the remediation either to maintain navigational depths or to remove contaminated sediments, deep cores or borings could be collected in the slips of Areas A, B, and C. Example core locations are shown on Figure 47. Pairs of cores could be collected in each slip in Areas A, B, and C. In addition, one core could be located near the S. Washington St. outfall to determine potential historical inputs and one core located near the fire dock to determine the depth of the high PAH concentration layer found in core WSF-A. Any existing boring logs from pier construction projects should be consulted to determine the depth to native material and plan the borings to include a sample from the native material.

5.3   OTHER DATA GAPS

Although this section focuses on the chemistry data gaps, other site information may also be useful for completing the site investigation and evaluating remedial alternatives. For example, biological testing might be useful in delineating cleanup boundaries south of Colman Dock where the chemistry data show mercury CSL exceedences extending out to at least the 50-foot depth contour. This also potentially holds true for northern boundary at Waterfront Park.

Detailed bathymetry was also lacking for the study area. This information is essential for planning and designing remedial alternatives and for modeling prop wash scour potential as recommended in Section 4.2. Therefore, the Panel authorized funds to conduct a bathymetric and pier survey as part of this study. The results are discussed in Section 6.1.

6.0 REMEDIAL DESIGN SUPPORT

6.1 BATHYMETRY

As discussed in Section 5.3, a new bathymetric map of the Elliott Bay Waterfront was needed for remedial design support. Existing maps prepared by the U. S. Government (U. S. Geological Survey or the National Ocean Survey) used contour intervals that were too large to be of value to the project team in providing input to the propeller wash modeling effort or to determine the geometry of any remedial actions. Existing maps were also not up to date in terms of recent changes along the Seattle Waterfront.

A bathymetic survey was conducted by ENSR Consulting and Engineering of Redmond, Washington to provide the project team with a current map of the study area. The survey was performed in mid-March 1995. The survey encompassed an area bounded on the north by the south face of Pier 62, on the east by the Alaska Way seawall, on the south by a line 500 feet south of the north face of Pier 46, and on the west by the 100-foot contour. The survey included preparation of a bathymetric map and identification of obstructions to dredging and capping under the piers.

6.1.1 Methods and Quality Assurance

The bathymetric survey was conducted from a 26-foot long aluminum survey vessel (Mary H) to U. S. Army Corps of Engineers Class 1 survey standards. Positioning was provided by a differential Trimble GPS system interfaced to an onboard computer with custom navigation software (HYPACK Ver 4.16). The software provided real-time horizontal positioning on the survey vessel with a position accuracy of 1 meter. Vertical data from an Innerspace precision fathometer was collected on an analog strip chart at the same time that depths were collected digitally and stored in the navigation computer every 3 to 6 feet along the survey trackline. Tracklines outside the piers were run on 100-foot centers, and tracklines inside the piers were run on 50-foot centers. Bar checks were done before and after each day of the field survey to check and calibrate the fathometer.

Post-survey data processing included correcting water depths to mean lower low water (MLLW), smoothing spiked data points, producing an ASCII digital data file, and printing check plots of the bathymetric map. Tide data from the NOAA tide gage at Pier 52 was used to reduce the survey depths to MLLW.

The under-pier survey included collecting spot depths under the piers, documenting conditions using a video-camera, and noting the location of piles and other obstructions. A fathometer was used from a Zodiac, aluminum skiff, and a canoe to collect the depth data. Survey lines were run using compass bearings from horizontal control points on the piers. Piling size and location and

the video surveys were done separately from the Zodiac.

A quality assurance (QA) program was performed by ENSR. Horizontal tracklines were plotted on-screen to check for data consistency and coverage. Digital data were checked against the analog depth chart and discrepancies were resolved or eliminated. Bar checks at three depths were taken at the beginning and end of each survey day. The azimuths of under-pier survey lines were verified from known positions on the opposite side of the pier at regular intervals. Vertical depth data were correlated to the NOAA tide station data and reduced to MLLW datum. Tie lines were run 30 feet and 80 feet off the pierhead line to cross-check spot depths, and were run under the piers when conditions permitted. Cross-lines were also run between piers. All of the data were plotted on a drawing at a scale of 1" = 50' as a final check. Any inconsistencies or misinterpreted piling offsets were corrected against GPS survey positions. The map was replotted and checked again.

6.1.2   Results

Figure 44, discussed previously in Section 3.4, gives a small-scale version of the resulting bathymetric map with contoured elevations relative to mean lower low water (MLLW) datum. A survey digital data file in ASCII format was used to produce the contour map. This file provides spot depths in MLLW and horizontal position in State Plane Coordinates, North Zone, NAD 83 datum.

Under-pier access is frustrated in many cases by wave curtains, curtain walls, interior bracing, offset piling, and pipes hanging from under the piers. A summary of the under-pier survey and its findings is presented in Appendix B. Appendix B also presents drawings of under-pier pile placement based on the under-pier survey.

6.2   PROPELLER WASH SCOUR POTENTIAL

A sand cap is one of the alternatives under consideration for remediation along the Seattle Waterfront. However, the conceptual site model showed that it would be important to evaluate the potential effects of prop wash prior to placing a cap. This section provides an estimate of the potential for cap scour from prop wash.

For the purposes of this evaluation, the cap was assumed to have a nominal thickness of three (3) feet as recommended by the US Army Corps of Engineers (A. Sumeri, personal communication, 1995). The average grain size of the cap material was assumed to 0.40 mm, based on grain size analysis results of Corps samples from material likely to be dredged and used as capping material. To determine the potential for scour of a sand remediation cap, a mathematical model was used to estimate the maximum depth of scour of the cap due to propeller wash from a variety of vessels known to use the Seattle Waterfront.

6.2.1 Model Methodology

Several models of propeller wash scour were screened for this report. These included models by Liou and Herbich (1976), Blaauw and van de Kaa (1978), Fuehrer et al. (1987), Hamill (1987), and Verhey (1983). All of the models, except for Hamill's, used momentum theory and the laws of free jet turbulence based on the pioneering work of Albertson et al. (1948) for submerged jets. The modelers idealized the propeller flow into that of a circular submerged jet and assumed that flow is steady, uniform, and frictionless.

All of the authors calibrated their propeller jet algorithms using results of scale model tests of the velocities within the propeller jet. All models assume that the diffusion process is dynamically similar and that the maximum axial velocity occurs along the axis of the propeller jet. Computed velocities within and at the boundaries of the propeller jet were compared to those from the scale model tests to adjust coefficients in the algorithms. Of special interest was the velocity of the propeller jet where it contacts the bottom boundary, herein referred to as the propeller or prop wash. The prop wash was combined with generally accepted equations for sediment incipient motion and transport to produce an algorithm for maximum depth of scour.

Hamill's model used two sizes of model propellers to study the scour pattern in a laboratory bed composed of two sizes of sand. A multivariate regression analysis of the results was used to derive equations relating the maximum depth of scour, and rate of scour development, to propeller and bottom characteristics such as propeller diameter, propeller speed, tip clearance, duration of propeller action, and bottom sediment size. Unfortunately, his results were in dimensional form, which did not allow a scaling up of parameters from the model scale to the larger real world scales for vessels using the Seattle Waterfront.

All of the other models use similar simplified algorithms to calculate the velocity distribution within the propeller jet. They all assume a normal or Gaussian probability distribution of velocities around the maximum axial velocity. None of the models take into account friction in the slipstream, vortex shedding off the propeller tips, and asymmetric flow within the jet. All but one of the models (Liou and Herbich, 1976) dealt with ducted and non-ducted single propeller configurations and accounted for boundary effects in their models. Verhey (1983) and Fuehrer et al. (1987) accounted for the presence of a center mounted rudder and included a coefficient for a twin propeller configuration. Given the simplifying assumptions in developing these models, their results must be considered simplified approximations of a complex process and resultant velocity field.

Of the prop wash models reviewed, the Hamill (1987) model was eliminated from the selection because of our inability to scale the model results up to the dimensions encountered along the waterfront. The Liou and Herbich (1976) model was also eliminated because of its simpler theoretical underpinnings.

Of the remaining prop wash models, the model chosen was based on the equations from Blaauw

and van de Kaa (1978) and Verhey (1983). The Verhey (1983) model is an extension of the Blaauw and van de Kaa (1978) model, both of which were developed at the Delft Hydraulics Laboratory, Delft, the Netherlands. The model will be referred to in the rest of this report as the Dutch model. The model by Fuehrer et al. (1987) (the Fuehrer model) is based on algorithms similar to the Dutch model but with different coefficients. In spite of their similarities, the Fuehrer model was not chosen to estimate scour potential for reasons discussed below.

The differences between the Fuehrer model and the Dutch model lie in the two primary coefficients which determine the location of the maximum velocity (the "c" coefficient), and the rate of decay of the maximum velocity downstream of the propellers (the "b" coefficient). The Fuehrer model uses an empirical exponential relationship which is a function of depth for the "c" coefficient, while the Dutch model uses a constant "c" coefficient with a slightly different value for ducted and nonducted propeller configurations. The "b" coefficient for the single propeller configuration is similar for each model, but for twin propellers is significantly lower (0.25) in the Fuehrer model than in the Dutch model (0.7).

In testing these two models, we found that in shallow water (e.g., propeller pointed toward the seawall in less than 20 feet), the bottom velocity computed by the Fuehrer model tended to increase at about five to ten propeller diameters behind the propeller, which was not seen in the Dutch model. It was suspected that the Fuehrer model "c" coefficient, which was a function of depth, was possibly causing the higher bottom velocities in shallow water.

To check the validity of the Fuehrer model "c" coefficient in shallow water, the reference papers containing the coefficient derivations were obtained. Also, Mr. Steve Maynord from the Corps of Engineers Waterways Experiment Station (WES) was contacted regarding their experience in applying the same prop wash models and coefficients. The outcome of this research was that: 1) the source of the "b" coefficient for the twin propeller case in the Fuehrer model could not be verified; and 2) Mr. Maynord's review of prop wash models concluded that the Dutch model had fewer empirical coefficients and relied more heavily on direct application of turbulent jet theory (Steve Maynord, personal communication, 1995). The Dutch prop wash model was chosen by WES in their study of the velocities induced by navigation along the Ohio River (Maynord, 1990). Mr. Maynord also pointed out that the empirical coefficients in the Fuehrer model were derived using a limited number of measured data points, and that he was not surprised that they gave less reliable results in shallow water. However, it should be noted that this does not imply that the bottom velocities calculated using the Fuehrer model in Section 3.3.4 are any less valid, since those results were calculated in water depths of equal to or greater than 30 feet.

The application of the Dutch model to multiple propellers was also tested by WES. As part of their scale model studies, the Dutch model was used to predict bottom velocities behind a twin propeller tow boat. The velocity behind the twin propellers was assumed to be additive in the region between the propellers, with the restriction that the total velocity could not exceed the maximum velocity of the single propeller. The measured and predicted bottom velocity distribution behind the tow boat were in good agreement (Maynord, 1990).

Based on our difficulty in applying the Fuehrer model in shallow water (i.e. less than 20 feet), and WES's success in applying the Dutch model, we choose to use the Dutch model to estimate scour potential.

6.2.2 Operational Areas

To assess scour potential for the remediation cap, the Seattle Waterfront was divided into seven vessel operation areas (Figure 48).

Operation Area 1 includes the slip between Piers 46 and 48. The types of vessels that have historically docked at the Pier 46/48 Slip include the passenger-only ferry, fish trawlers, tugboats, and US Navy ships. An exaggerated profile of the situation is shown on Figure 49. The profile assumes that all of the vessels are docked with their stern and propellers pointed seaward or toward the west. This is a reasonable assumption since most vessel captains do not want to chance damaging their propellers by hitting the shallower bottoms at the head end of pier slips, and they usually want the propeller in deeper water when it comes time to maneuver away from the pier. One exception to this might be the tug. With a shallow draft and high maneuverability, a tug might turn its stern toward the head of the slip.

Operation Area 2 includes the area between Piers 48 and 52. Trawlers and the Canadian auto ferry *Royal Victorian* dock at Pier 48. The WDOT passenger-only ferries dock at the pier jutting diagonally toward Pier 48. The trawler and auto ferry propeller configurations are shown in exaggerated profile on Figure 50. and the passenger-only ferry configuration on Figure 51.

Operation Area 3 is the Seattle Ferry Terminal at Pier 52. Only WDOT super-ferries of the Washington State Department of Transportation operate in this area. The ferries have a propeller at each end of the vessel. As shown in the exaggerated profile in Figure 52, we modeled the propeller pointing or acting toward Pier 52. When the ferry is approaching the pier for docking, we assume this is the propeller that would be used to slow the ferry down. During ferry departure from the terminal, this is also assumed to be the propeller that would be used to get underway.

Operation Area 4 is the Seattle Fire Dock. Both the *Chief Seattle* and the *Alki* fireboats are moored stern to the dock. The situation is shown in profile on Figure 53.

Pier 55, the harbor tours dock, constitutes Operation Area 5. The *Spirit of Seattle* and the *Good Times* are typical of the boats that use this area. The sightseeing boats usually dock with their stern pointed seaward. However, to simulate worst-case conditions, we modeled them in the opposite direction with their sterns pointed toward Alaskan Way. This situation is shown on Figure 54.

Operation Area 6 is the slip between Piers 56 and 57. The tour boat *Sightseer* and an occasional fishing boat are typically seen docked here. The *Sightseer* can dock either with her stern

pointing shoreward, seaward, or across the slip between Piers 56 and 57. The worst case would be with the stern pointing shoreward, which was used in the analysis. A fishing boat may, and probably would, dock with her stern pointed toward the Alaska Way seawall. This situation is shown in Figure 55.

Operation Area 7 is the open area between Piers 57 and 59 and in front of the Waterfront Park on Pier 58. Vessels use this area infrequently, and only small pleasure boats use the area north of Pier 57. Occasionally, tugs will berth on the end of Pier 57 (Clark, personal communication, 1995).

6.2.3 Modeling

The Dutch model was chosen to provide an estimate of the maximum depth of scour to a cap from vessels in each operation area along the Seattle Waterfront. Generic ship characteristics and site conditions are input for each vessel operation area. The prop wash computation sequence includes calculation of propeller efflux velocity, axis velocity, radial velocity, and bottom velocity as a function of distance behind the propeller. The axis velocity and sediment characteristics are used to calculate a Froude number, which is a dimensionless ratio of inertial to gravitational forces. The Froude number, along with propeller height and diameter, is used to calculate the maximum depth scour using empirical coefficients based on results of scour tests (Verhey, 1983).

Propeller efflux velocity (V_o), or the initial axial velocity of the propeller jet a half propeller distance behind the plane of the propeller, is calculated in both models according to the following equation:

$$V_o = 1.6nD(K_t)^{0.5},$$

where: n = propeller rotational speed,
  D = propeller diameter,
  K_t = propeller thrust coefficient ($0.35\pm20\%$ from Fuehrer et al., 1987).

The rest of the equations leading to the determination of scour depth appear in dimensionless form, which with consistent units should allow scaling the results from model to prototype scales. The axis velocity ($V_{x,0}$) any distance "x" behind the propeller is given by:

$$V_{x,0}/V_o = (D_o/2cx)^b,$$

where: D_o = initial jet diameter,
  = D for a ducted propeller,
  = $D/(2)^{0.5}$ for a nonducted or open propeller,
  c = empirical coefficient,
  = 0.17 for a ducted propeller,

= 0.19 for a nonducted propeller, but the average 0.18 is recommended
by Verhey (1983) and used herein,

x = axial distance behind the propeller,

b = empirical coefficient,

= 1.0 for no rudder,

= 1.1 with a central rudder,

= 0.7 with multiple rudders.

The radial velocity ($V_{x,r}$), or velocity in the jet a distance "x" behind the propeller and a distance "r" from the propeller axis, is computed using the radial distance (r) in equation:

$$V_{x,r}/V_{x,0} = \exp[-r^2/2c^2x^2].$$

where: r = radial distance from the propeller axis.

If the distance from the propeller axis to the bottom (z) is substituted for the radial distance, the scour velocity due to the propeller jet or the propeller wash ($V_{x,z}$), is given by:

$$V_{x,z}/V_o = 2.78(D_o/x)\exp[-15.43z^2/x^2],$$

where: z = depth from propeller axis to bottom,

= h-s

h = water depth

s = propeller shaft depth.

The maximum scour depth (S_{max}) was derived empirically by Verhey (1983) and is given as:

$$S_{max}/z = 4\times10^{-3}(FD_o/z)^{2.9},$$

where: $F = V_{x,z}/(g\Delta d_{50})^{0.5}$, a jet Froude number

g = gravitational acceleration

$\Delta = [(\rho_s-\rho_w)/\rho_w]$, relative density of the bottom sediments

d_{50} = median bottom grain size diameter

ρ_s = sediment density

ρ_w = water density.

The only limitations stated in the literature were that for the above equations to be valid, the axial distance had to be twice the propeller distance (x>2D) and the depth had to be between 90 and 900 percent of the propeller diameter (0.9<h/D<9).

The equations were entered onto a spreadsheet for tabular computations using characteristics of the vessels calling at each operating area along the Seattle Waterfront (shown in Table 6-1). The vessel information provided in the table is from Francisco (1995). Single propeller vessels were

assumed to have a single central rudder and multiple propeller vessels were modeled with multiple rudders. Simulations were computed for a series of axial distances (usually 100-foot intervals) and corresponding depths behind each propeller for each operational scenario. Iterations were made to determine the axial distance and depth of the maximum bottom velocity and corresponding maximum scour depth. Computations were ended when the bottom rose to the elevation of the propeller axis (like along the shoreline in front of the seawall) or at an axial distance of 500 feet, whichever occurred first. In keeping with the way the models were developed, it was assumed the vessel was stationary or tied up to the dock during the simulations. The calculated scour was assumed to be in equilibrium with the propeller wash; that is, the calculated scour was assumed to be the maximum scour expected if the vessel sat at the dock and churned away until no more scour occurred. If the vessels producing scour are maneuvering or passing over the bottom, the scour equations will provide a conservative (read over-estimated or worst-case) estimate of the scour.

6.2.4   Model Results

The depth of scour was calculated for the proposed remediation cap at each operation area. Simulations were made at mean lower low tide using the vessel and area characteristics on Table 6-1. The results are summarized on Table 6-2.

Operation Area 1

The model results (see Appendix C for the spreadsheet tabulations) indicate that there is potential scour of about 1.3 feet from the trawler. Maximum scour occurs in an area about 125 feet behind the trawler. This is an area 400 feet to 500 feet seaward of the bulkhead. All other vessels would not scour at this location. The tug appears to be in water too deep and has a propeller too small to impact a sandy bottom, and the US Navy ship is in water too deep to have the bottom impacted by its propeller wash.

As a check against all reasonable possibilities, the tug was modeled with its stern facing toward the head of the slip. The model indicates that a tug could scour approximately 0.5 feet between 50 feet and 100 feet seaward of the bulkhead.

Operation Area 2

The model results suggest that under normal operating conditions only the trawler would have the potential for scouring the sand cap. The depth of scour may be up to 1.6 feet for a trawler. The bulk of the scour would occur in an area 400 feet to 450 feet seaward of the bulkhead.

To address all possibilities, the auto ferry *Royal Victorian* was modeled with its propellers in reverse, i.e., directed landward, and also the bowthruster was modeled. The simulations suggest that the bowthruster would not scour a sand cap placed below elevation -15 feet MLLW. The main propellers in reverse might scour a swath up to 0.5 feet deep into a sand cap from the

bulkhead to about 400 feet seaward.

Table 6-1. Vessel Operation Scenarios. Data from Francisco (1995)

VESSEL OPERATION AREAS	VESSELS	VESSEL CHARACTERISTICS			OPERATION CHARACTERISTICS
		PROPELLER DIAMETER FT.	SHAFT DEPTH FT.	SHIP LENGTH FT.	DOCKING SHAFT RPM
1	Tugboat	6	5	80	60
1	Trawler	14	12	120	60
1	US Navy Ship	15 - Twin	20	600	50
1	Passenger Ferry - *TYEE*	3.6 - Twin	5	86	360
2	Canadian Ferry - *ROYAL VICTORIAN*	8 - Twin*	7	450	100
2	Passenger Ferry - *TYEE*	3.6 - Twin	5	86	360
2	Trawler	14	13	120	60
3	Super Ferry - Idle	12	12	382	50
3	Super Ferry - Underway	12	12	382	140
4	Fireboat - *CHIEF SEATTLE*	3.5 - Triple	3	96	180
4	Fireboat - *ALKI*	4 -Twin	3	123	180
5	*SPIRIT OF SEATTLE*	4.7 -Twin*	4	115	180
5	*GOOD TIMES*	3.7 - Twin	3	87	180
6	*SIGHTSEER*	3.7	3	35	180
6	Fishing Boat	3	3	50	300
7	No Vessels	NA	NA	NA	NA

* Also has a bowthruster.

Operation Area 3

The model results indicate that when the super ferries are running at idle at the terminal, the propeller wash could scour up to 0.2 feet into a sand cap. Only if the ferry were tied to the dock using its cruising propeller speed, would it potentially scour through the cap.

Table 6-2. Maximum Depth of Scour (in feet) for Each Operation Scenario from the Dutch Model.

VESSEL OPERATION AREAS	VESSELS	PROPELLER CONFIGURATION	SCOUR DEPTH (Feet)	ASSOCIATED WATER DEPTH (Feet)
1	Tugboat	Single	0.5*	8
1	Trawler	Single	1.3	35
1	USNS Ship	Twin	0	78+
1	Passenger Ferry - *TYEE*	Twin	0	35
2	Canadian Ferry - *ROYAL VICTORIAN*	Twin	0.5*	26
2	Passenger Ferry - *TYEE*	Twin	0	44
2	Trawler	Single	1.6	34
3	Super Ferry - Idle	Single	0.2*	33
3	Super Ferry - Underway	Single	7.2*	32
4	Fireboat - *CHIEF SEATTLE*	Triple	0.4*	10
4	Fireboat - *ALKI*	Twin	1.1*	10
5	*SPIRIT OF SEATTLE*	Twin*	1.0*	12
5	*GOOD TIMES*	Twin	0.1*	15
6	*SIGHTSEER*	Single	0*	8+
6	Fishing Boat	Single	0.5*	4
7	No Vessels	NA	NA	NA

* Propeller jet directed toward the Alaskan Way Seawall.

Assuming a cruising propeller speed when the ferries depart, or when the ferries use the bow propeller to slow for arrivals, there is potential for scouring a sand cap under the terminal. Note that the sand cap, with an average grain size of 0.4 mm, is finer than the existing material under the older northern portion of the ferry terminal, which range in mean grain size from 0.5 mm to 1.0 mm (see Figure XX). This supports the model prediction that the 0.4 mm grain size cap material under the terminal may eventually be scoured.

Operation Area 4

The model results suggest that the *Chief Seattle* could scour up to 0.4 feet and the *Alki* up to 1.1 feet about 100 feet behind the fire boats. The scour would be near the seawall.

Operation Area 5

Modeling results suggest that the *Good Times* might erode a sand cap at Pier 55 to a depth of 0.1 feet. The *Spirit of Seattle*, however, might scour to a depth of 1.0 foot immediately in front of the seawall. The depth under Pier 55 next to the seawall is estimated to be about 12 feet. This is based on depths next to the seawall under Pier 56 which were 16 feet, since the bathymetric survey did not obtain depths at the seawall under Pier 55.

Operation Area 6

The model indicates that the *Sightseer* might not scour a sand cap. A fishing boat might not scour under normal docking procedures but may scour up to 0.5 feet of sand cap with its stern pointed toward the seawall assuming a depth of 4 feet. The scour area would include the area from the seawall out about 30 feet to 60 feet. However, this estimate assumes a depth out to 60 feet between Piers 56/57 since this area was not accessible during the bathymetric survey.

Operation Area 7

Vessels observed in this area have propellers that are too small, or speeds too slow, or the water is too deep for scour to occur.

6.2.5 Discussion of Results

As mentioned above, the models use simplified methods to simulate complex processes and, as a consequence, the model results must be interpreted carefully. The results presented above are from simulations that can be divided into two computational processes: the calculation of the propeller jet velocities, and the scour calculations based on the computed bottom velocities. It is important to note again that the results of the simulations are based on scale modeling that was carried to equilibrium conditions. Therefore, the simulation results assume that a vessel would sit in the operation area and churn until the propeller wash and scour depth reached equilibrium for the input conditions. Thus, propeller wash velocities and scour depth values represent "worst-case" results.

The first process, calculation of propeller jet velocities and propeller wash, seems to relate well to real-world experiences. Propeller wash velocities rarely exceeded 200 cm/sec, and then only with the propeller jet directed toward a rising bottom. Based on the discussion in Section 3.2 above, some scour of a sand cap would be expected when the bottom velocity from the propeller exceeds 25 to 30 cm/sec. The propeller wash or bottom velocity equations for a single propeller with a central rudder was given by:

$$V_{x,z} = 3.14nD^2K^{0.5}x^{-1}exp(-15.43z^2/x^2).$$

This equation suggests that the propeller wash is decreasingly sensitive to propeller diameter,

propeller speed, distance behind the propeller, and thrust coefficient. A sensitivity analysis (Appendix C) for the modeled bottom velocity based on the above equations yielded the following:

- a 10 percent change in propeller diameter causes about a 13 to 21 percent change to bottom velocity

- a 10 percent change in propeller speed causes about a 10 percent change to bottom velocity

- a 10 percent change only in distance behind the propeller causes about a 3 to 10 percent change in bottom velocity

- a 10 percent change in thrust coefficient only causes about a 5 percent change in bottom velocity.

The sensitivity analysis shows that the propeller jet velocity model is sensitive to propeller diameter and propeller speed, and less sensitive to distance behind the propeller and thrust coefficient. Propeller diameters and thrust coefficients are known to about 10-20%, while propeller speed and distance behind the propeller are assumed constant for each scenario. Therefore, we can estimate that error from uncertainty in the input parameters to the propeller jet velocity model is on the order of 25-50%.

The second process, the calculation of scour depth, was assessed by looking at all the input parameters combining the propeller jet velocity equations with the maximum scour depth equation. The resulting equation for the Dutch model is given by:

$$S_{max} = 0.04z^{-1.9}n^{2.9}D^{8.7}K^{1.45}x^{-2.9}(g\Delta d50)^{-1.45}(exp[-15.43z^2/x^2])^{2.9}.$$

The exponential factors in the equation indicates that modeled scour depth is decreasingly sensitive to propeller diameter, propeller speed, bottom depth, distance behind the propeller, and thrust coefficient. A sensitivity test using the equation gave the following:

- a 10 percent change in propeller diameter causes about an 81 to 129 percent change to scour depth

- a 10 percent change in propeller speed causes about a 32 percent change to scour depth

- a 10 percent change in bottom depth only causes about a 20 percent change in scour depth

- a 10 percent change only in distance behind the propeller causes about a 7 to 32 percent change in scour depth

- 71 -

- a 10 percent change in thrust coefficient only causes about a 15 percent change in scour depth

- a 10 percent change in grain diameter only causes about a 13 to 17 percent change in scour depth.

The sensitivity analysis shows that the scour model is sensitive to all of the input parameters and highly sensitive to the propeller diameter. Given that propeller diameters and thrust coefficients are known to about 10-20%, the estimated mean grain size is known to about 50%, while propeller speed and distance behind the propeller are assumed constant for each scenario, we can estimate that error from uncertainty in the input parameters to the scour depth model is on the order of 225-700%.

Without actual field measurements to verify the model simulations, the only recourse available to appraise the output was to compare the output under varying conditions (see Appendix C). There was concern when analyzing the results that the conditions being simulated along the Seattle Waterfront may be beyond the limits for which the scour model was developed. Unfortunately, the literature gave no guidance as to what the limits might be. We do know that the scour depth equation is based on the original tests carried out by Rajaratnam (1981) for Froude numbers between 5.1 and 5.4. The Froude numbers in our scenarios are generally less than 8.

6.2.5   Conclusion and Recommendations

Given the uncertainties in the scour depth model, the maximum scour depth results should be viewed as a qualitative estimate rather than as a strict quantitative estimate. With this caveat in mind, the model results suggest that some scour may occur in a 3-foot thick remediation cap of sand placed along the Seattle Waterfront in at depths of less than about 40 feet.

A remediation cap or caps placed in the area from Pier 46 to Pier 57 should be monitored periodically to determine if maintenance or armoring is will be required. The scour as modeled does not seem to be serious enough to warrant placement of a gravel armor layer initially. Additional sand may be needed to return the minimally scoured areas to original conditions or gravel may be needed to armor the areas if scour exceeds 1 foot or if incidences of scour are likely to be repeated.

6.3   CAPPING/ARMORING SOURCES AND MATERIALS

6.3.1   Selection of Capping Material

Capping is the controlled placement of a covering or cap of clean isolating material over contaminated material. The term "contaminated" refers to material that exceeds the Cleanup Screening Levels listed in the Sediment Management Standards, Chapter 173-204 WAC, while

the term "clean" refers to material that meets the long-term goal for sediment quality in Puget Sound, or the Sediment Quality Standards. Materials suitable for capping of contaminated sediment can be divided into three categories: inert, chemically active, and sealing agents. To date, capping of contaminated sediment has been done almost exclusively with clean, inert material such as sand and silt.

Site specific biological populations need to be considered when selecting inert capping material and designing cap thickness. One objective of capping is to isolate the contaminated sediment from benthic organisms. A sand cap will attract suspension-feeding organisms which are not deep burrowers, while a fine-grained cap will attract deep burrowing feeders. Bokuniewicz (1986) reported that a sediment cap thickness of 3 feet is generally adequate for relatively protected nearshore waters, but site-specific studies should be done to evaluate erosion potential and biological populations. The Corps of Engineers, Seattle District, also recommends a cap thickness of 3 feet to isolate the contaminated material.

An additional consideration for selection of capping material, cap thickness, and need for armoring is the erodibility of the cap. Fine sands are generally more erodible than coarse sands or cohesive material. Cohesive capping materials excavated by mechanical dredges are more erosion resistant than those excavated by hydraulic dredges.

As stated in Section 6.2, the prop wash scour potential modeling suggests that the cap does not appear to warrant use of armoring material. However, sources and costs for both capping and armoring will be briefly discussed in case monitoring shows that armoring material is required.

6.3.2 Capping Material Sources

Sources of capping material were identified through a survey of:

- Seattle District Corps of Engineers - available material from future maintenance dredging projects.

- Other non-Corps projects identified through the PSDDA Dredged Material Management Office (Stirling, personal communication, 1995).

- Local ports.

- Beneficial Uses Work Group (Sumeri, personal communication, 1995).

- Local sand and gravel operations.

- Local dredging contractors.

There are several local sources for sand as capping material, including commercial sites, dredged

- 73 -

material from Corps of Engineers (COE) maintenance dredging projects, and other non-COE projects. The commercial sites are owned and operated by Lone Star Northwest and are located in Steilacoom and on Maury Island. The COE maintenance dredging projects are located in the Duwamish Waterway, in the Snohomish River, and in the Swinomish Channel. Other non-COE projects include Capital Lake, in Olympia, and U.S. Navy Norton Terminal, in Everett.

The Washington State Department of Natural Resources (DNR) has the responsibility for managing the removal of valuable materials from the bed of state owned aquatic lands. DNR should be contacted to asses the ownership of sediments from state owned aquatic lands, prior to the removal of dredged material for beneficial use.

The Steilacoom and Maury Island sites can provide sand and gravel for capping material. Cost of sand is approximately $2.50 per ton, with another $1.00 - $1.25 per cubic yard (cy) to haul to project site. Total cost to purchase and haul sand is approximately $4.35 - $4.60 per cy. Cost of gravel is approximately $5.00 per ton. Therefore cost to purchase and haul gravel is approximately $8.00 - $8.25 per cy.

Another option is to obtain silty sand material from the maintenance dredging of Duwamish Waterway. The volume of available material from this site is approximately 50,000 to 60,000 cy every other year, consisting of approximately 24,000 to 40,000 cy of sand material. There is no cost to purchase the sand, the only cost is from hauling the material to the project site. Duwamish Waterway dredged material could be used for this project at a hauling cost of approximately $1.00 - $1.25 per cy (Sumeri, personal communication, 1995).

Sand material can also be obtained from the maintenance dredging of the upstream basin in the Snohomish River. The volume of available material from this site is approximately 250,000 cy every other year. The DNR Contaminated Sediments Management Section should be consulted on the beneficial use of Snohomish River sediments for capping purposes. The cost for hauling Snohomish River dredged material to the Eagle Harbor site was $1.00 - $1.25 per cy (Parker, personal communication, 1995). Gravel is also not available at this site. Silty sand material is available from maintenance dredging in the downstream basin in the Snohomish River. Available volume is approximately 200,000 cy every four years. Cost to haul would be the same as for the upstream basin.

Sand material is also available from the Swinomish Channel (Parker, personal communication, 1995). Approximately 100,000 cy are excavated every three years. Material from the Swinomish Channel was used for the Grays Harbor project at a haul cost of approximately $3.00 to $4.00 per cy.

The Capital Lake project in Olympia has approximately 30,000 cy of sand available. Hauling costs from the site would be approximately $2.00 to $2.50 per cy. Project start is scheduled for late 1996 to early 1997. Preliminary testing has identified this material to be suitable for unconfined open water disposal (Hong West, 1994).

The U.S. Navy Norton Terminal in Everett will require dredging of approximately 100,000 cy of silty sand. The material has been tested and found suitable for unconfined open water disposal (Stirling, personal communication, 1995). Cost to haul material from this site would be comparable to the cost to haul Snohomish River material at approximately $1.00 - $1.25 per cy. This project is not currently scheduled.

Table 6-3   Sources of Capping Material, Material Type, Available volumes, Year Available, and Approximate Cost

SOURCE	MATERIAL TYPE	VOLUME (CY)	YEAR AVAIL.	$/CY (ON-SITE)
Lone Star Northwest	Sand	As req'd	any year	$4.35-$4.60
Lone Star Northwest	Gravel	As req'd	any year	$8.00-$8.25
Duwamish, Upstream Basin	Silty Sand	60,000	1996	$1.00-$1.25
Duwamish, Upstream Basin	Silty Sand	50,000	1998	$1.00-$1.25
Duwamish, Upstream Basin	Silty Sand	50,000	2000	$1.00-$1.25
Snohomish, Upstream Basin	Sand	250,000	1995	$1.00-$1.25
Snohomish, Upstream Basin	Sand	250,000	1997	$1.00-$1.25
Snohomish, Upstream Basin	Sand	250,000	1999	$1.00-$1.25
Snohomish, Downstream Basin	Silty Sand	200,000	1996	$1.00-$1.25
Snohomish, Downstream Basin	Silty Sand	200,000	2000	$1.00-$1.25
Swinomish Channel	Sand	100,000	1997	$3.00-$4.00
Swinomish Channel	Sand	100,000	2000	$3.00-$4.00
Capital Lake	Sand	30,000	1996-97	$2.00-$2.50
U.S. Navy Norton Terminal	Silty Sand	100,000	?	$1.00-$1.25

There are also miscellaneous projects each year that have clean material available. The DNR estimates that there will be a total of approximately 90,000 cy of dredged material suitable for open water disposal from projects in the Puget Sound for fiscal year 1996 (Sweeney, personal communication, 1995). However, the type of material may vary from year to year.

Table 6-3 summarizing sources of capping material, material type, available volumes, year available, and approximate cost is provided below.

6.4 UNDER PIER AND OPEN WATER REMEDIATION TECHNOLOGIES

There are two conventional methods of remediation for contaminated material. The first is to remove the contaminated material by dredging and disposal at an approved site. The second is to isolate the contaminated material from the aquatic environment by placing a clean sediment cap on top of the contaminated material. A third method of remediation is to first remove the contaminated material then treat it. Any of the three methods of remediation can be used in combination with any other method to provide the best remediation alternative. This section discusses feasible alternatives for remediation of the areas both under the piers and in open water.

6.4.1 Under Pier Remediation Technologies

Due to the confined spacing under the piers, it will not be possible to fit large equipment under the piers. If spacing between piling is fairly uniform, it will be possible to extend either a pipeline or conveyor system under the pier to either dredge or cap the contaminated material. However, if spacing is not uniform, or there is limited access due to bumper piling, there will be greater difficulty in either dredging or capping which may significantly increase costs. The piers that may have significant access difficulties include: Piers 46, 52, 54, 57, and 59 (see Appendix B). Structural stability of existing pilings and docks also needs to be addressed during the design of the remediation alternatives under the piers.

6.4.1.1 Capping

Use of appropriate equipment and placement techniques for the capping material is a critical requirement for any capping operation. Placement of capping material should be accomplished so that the deposit forms a layer of required thickness over the contaminated material. A nominal sand cap may be three feet thick, but can be less in areas designated for enhanced natural recovery. The equipment and placement technique should be selected and rate of application controlled to avoid displacement of the contaminated material. Since water column dispersion of capping material is not usually of concern, the use of submerged discharge for capping placement should only be considered from the standpoint of controlling the placement (Palermo, 1991a).

There are two methods available for capping under piers: hydraulic and mechanical. The hydraulic option would involve barging in the cap material, then slurrying up the cap material in order to transport the slurried cap material through a discharge pipeline on floats. The discharge would be at the surface unless better control of the cap placement were required; submerged discharge would then be an option. A variation of this method is to use a concrete pumper, which is capable of transporting a high solids content slurry. The cost to place the cap material will vary depending upon the volumes to be placed, due to the high cost of mobilization/demobilization. Based upon our experience and conversations with contractors, a typical cost range would be approximately $65/cy to $75/cy of cap material placed (Hillman,

personal communication, 1995); this does not include the cost to obtain cap material.

The mechanical option would be the use of a conveyor system (on floats or attached to the pier deck) to transport the cap material under the piers. Placement of the cap material would be directly from the end of the conveyor or pushed off the side of the conveyor. Cap material would have to be barged in from off-site, then a loader would supply the cap material to the conveyor system. As stated for the hydraulic option, the cost to place the cap material will vary depending upon the volumes to be placed. A typical cost range would be approximately $50/cy to $70/cy of cap material placed; this does not include the cost to obtain cap material (Hales, personal communication, 1995).

6.4.1.2 Dredging

Due to the confined spaces under piers, it would not be possible to utilize conventional dredging equipment such as a clamshell or cutterhead dredge. A diver assisted hydraulic dredging system has been successfully employed at several project sites, including the Sitcum Waterway Remediation Project and LaConner Marina. The cost to dredge contaminated material from under piers will vary, depending upon the volumes to be dredged (mobilization/demobilization costs are high), and the difficulty in access. The depth of dredging will be variable depending on the depth of contamination and the remediation method. Based upon our experience and conversations with contractors, a cost range for a typical volume of less than several thousand cubic yards would be approximately $70/cy to $100/cy. This would include under dock work, with the material in the barge and transported to a Puget Sound offloading site. The unit cost does not include the cost for rehandling, treatment, and final disposal of the contaminated material (Hillman, personal communication, 1995).

6.4.1.3 Removal and Treatment

There are several treatment options that can be applied after the contaminated material is removed from the aquatic environment. Treatment offers an alternative to disposal at an upland site and may allow beneficial use of the material after the treatment. The treatment methods briefly discussed in this report include: soil washing, bio-remediation, and thermal remediation.

Soil Washing

In this treatment method, contaminants absorbed onto soil particles are separated from the soil in an aqueous-based system. The wash water may be augmented with a basic leaching agent, surfactant, pH adjustment, or chelating agent to help remove organics or heavy metals. Soil washing is limited by the following factors: 1) fine soil particles (silts, clays) are difficult to remove from washing fluid, 2) complex waste mixtures (metals with organics) make formulating washing fluid difficult, and 3) high humic content in soil inhibits desorption. Soil washing costs average $50 to $100 per cy. This unit cost does not include the additional cost to dredge and transport the material to the treatment site. The dredging and transport cost depends on the

dredging and treatment site locations (EPA, 1993).

Bio-Remediation

In bio-remediation, excavated soils and sediments are mixed with soil amendments and placed in above-ground enclosures that include leachate collection systems and some form of aeration. Biodegradation is enhanced with nutrients, oxygen, and pH in the bioreactor. Factors which affect applicability and effectiveness include: 1) large amounts of space are required, 2) depending upon the contaminant, the process can require considerable amount of time, 3) sediment may require some dewatering prior to treatment, and 4) process is sensitive to oxygenation and nutrient loading rates. Bio-remediation costs average $50 to $100 per cy. This unit cost does not include the additional cost to dredge and transport the material to the treatment site. The dredging and transport cost depends on the dredging and treatment site locations (EPA, 1993).

Thermal Remediation

In this treatment method, low temperature desorption processes vaporize the organic contaminants, which are then captured, condensed, and removed from site. The following factors may affect applicability and effectiveness: 1) debris or other large objects buried in the media can cause operating difficulties, 2) limited range of applicable contaminants.

The high temperature processes (greater than 1,600 °F), such as incineration, vitrification, and cement processing technologies, volatilize or combust the organic contaminants. Inorganic contaminants are either encapsulated in the vitrification/solidification technologies and isolated from the environment or remain in the ash for disposal from the incineration technologies. Applicability and effectiveness can be affected by the following factors: 1) feed size and materials handling requirements can make technology impractical, 2) volatile metals may affect performance or incinerator life, 3) some contaminants may leave the combustion unit with the flume gases, and 4) some metals can react with other elements in the feed stream and form more volatile and toxic compounds than the original contaminants.

The low temperature thermal processes average $40 to $75 per cy while the high temperature technologies cost $100 to $300 per cy. This unit cost does not include the additional cost to dredge and transport the material to the treatment site or the disposal of residual waste streams. The dredging and transport cost depends on the dredging and treatment site locations (EPA, 1993; Case personal communication, 1995).

6.4.2   Open Water Remediation Technologies

The remediation alternatives for open water are the same as for under pier: capping, dredging, and removal and treatment. Capping and dredging will be discussed further since methods will vary from under pier work. Treatment options and cost will remain the same as for under pier

work. Remediation in open water areas is less complex than under the piers, however site specific issues are still critical in determining probable costs.

One important issue is the need to provide for navigation in the open water areas. In a recent Puget Sound Water Quality Authority report (PTI, 1993), recommendations were provided for required depths for navigation and commerce needs along the Seattle central waterfront. The recommendations were to maintain existing water depths seaward of the inner harbor line or a draft of -30 ft from the inner to outer harbor line and -40 ft beyond outer harbor line.

Effectively, this recommendation means that in any area that capping is proposed, dredging equal to or greater than the capping depth would be recommended in order to prevent the decrease of water depth for navigation. This recommendation is intended to be used as reference when considering future actions in Elliott Bay and the Duwamish Estuary.

Another site specific issue is the confined working spaces between the piers. The confined spacing will limit the type and methods used for either capping or dredging.

6.4.2.1 Capping

A number of different equipment types and placement techniques can be used for capping operations. Surface discharge of cap material from barges, or from hydraulic pipelines (slurried cap material) can be used if anticipated bottom spread and water column dispersion are acceptable. If unacceptable or greater control is required, then equipment such as diffusers, tremies, or other equipment for submerged discharge can be used. The compatibility of equipment and placement techniques, and accuracy and control of the equipment during placement, are essential for any capping project (Palermo, 1991b).

Conventional methods of cap placement may be insufficient to prevent contaminated material from being resuspended and/or bottom impact waves from occurring; other cap placement methods such as hosing off the cap material from the barge may be required to control cap placement. Some methods, such as cap placement using barge movement, are limited due to the confined spacing between piers. Based on our experience and conversations with contractors, a typical cost range for capping in the open water areas is approximately $5/cy to $10/cy of cap material placed; this does not include cost of the cap material (Hales ; Hillman, personal communication, 1995).

6.4.2.2 Dredging

Dredging in the open water areas can be accomplished using either mechanical or hydraulic dredging. The major advantage of using a mechanical system instead of a hydraulic system is that a mechanical system does not entrain the large volumes of water necessary for a hydraulic system. The water entrained using a hydraulic system would probably need to be treated before it could be returned to Elliott Bay, which raises the cost per cubic yard.

Control of dredge depth is generally better with a hydraulic system, however, new mechanical buckets (i.e., cable arm closed bucket) are providing improved control to plus or minus one feet, which is as accurate as a hydraulic system.

The probable cost range for dredging was derived assuming that dredging would be accomplished using mechanical means. Based on our experience and conversations with contractors, a typical cost range for dredging in the open water areas is approximately $3/cy to $10/cy (Hales ; Hillman, personal communication, 1995), depending on the volumes dredged (i.e. mobilization and demobilization costs will affect unit cost). This does not include cost to rehandle ashore and dispose of material.

Table 6-4 Remediation Methods and Estimated Unit Costs

SUMMARY TABLE		
REMEDIATION METHOD	LOCATION	UNIT COST RANGE

Capping	Under Pier
	    - Concrete Pumper
	    - Conveyor
	Open Water
		
		$65-$75/cy[1]
		$50-$70/cy[1]
		$5-$10/cy[1]

Dredging	Under Pier - Diver Assisted Open Water	 $70-$100/cy[2] $3-$10/cy[2]

Treatment
    - Soil Wash
    - Bio-remediation
    - Thermal remediation
        Low temp.
        High temp.
	
	Both
	Both
	
	Both
	Both
		
		$50-$100/cy[3]
		$50-$100/cy[3]
		
		$40-$75/cy[3]
		$100-$300/cy[3]

Disposal
    - Rehandling Upland
    - Upland Tipping Fees
	
	N/A
	N/A
		
		$5-$7/cy
		$45-$50/ton

[1] Does not include the cost to purchase and haul capping material.
[2] Does not include disposal costs.
[3] Does not include contaminated material removal costs.

6.4.2.3 Removal and Treatment

The same treatment options as discussed in section 5.4.1.3 are available for contaminated material removed from the open water areas.

6.4.3 Disposal of Dredged Contaminated Material at an Approved Upland Site

The cost to dispose of contaminated material upland adds substantial cost to a dredging project. The contaminated material needs to be rehandled from the barge and transported to the upland disposal site, typically via a transfer station that uses rail for transport. The cost of rehandling and transporting the contaminated material to a transfer station is approximately $5/cy to $7/cy (Hales ; Hillman, personal communication, 1995). If the material requires dewatering, costs could be significantly higher.

A typical upland fee for contaminated material, which includes testing to determine suitability for disposal at the designated upland site, is approximately $45 to $50/ton at the Roosevelt Landfill (Mork, personal communication, 1995). Table 6-4 summarizes the unit costs for all the remediation methods discussed above.

7.0 SUMMARY AND CONCLUSIONS

The primary goal of the Elliott Bay Waterfront Recontamination Study was to determine if it is feasible for the Panel to undertake sediment cleanup projects along the Seattle central waterfront. Several study objectives were developed to answer this question, listed in Section 1.2. These objectives are reviewed below, along with the key findings of the study that answer the questions posed by the Panel.

Measure the rate of recontamination and determine the rate of sedimentation in the study area. Three components of the study were targeted at this objective - 1) particulate material falling to the bottom was captured in sediment traps along the waterfront and contaminant concentrations in this material measured, 2) core samples were collected and dated to evaluate the rate of sedimentation in the study area, and 3) a source control evaluation of ongoing discharges was conducted to evaluate whether source control is complete enough to begin remediation.

Upon analysis, material in the sediment traps was found to be contaminated above SMS cleanup screening levels. The distribution of contaminants among trap stations was very similar to chemical distributions in surface sediments. The most common contaminants were mercury, PAHs, phenols, and phthalates. Based on this finding, it was important to identify the sources of these contaminants, since it appeared that particulates in the water column could be a source of recontamination to a newly cleaned-up area.

The source control evaluation determined that (with the exception of the S. King St. CSO) none of the CSOs or storm drains along the waterfront discharge enough contaminants to cause recontamination above CSLs, nor do particulates carried to the waterfront by the Duwamish River appear to be contaminated above CSLs. Therefore, these sources are considered controlled and not likely to result in recontamination after cleanup to levels above cleanup standards. However, the S. King St. CSO remains as a possible source of localized contamination above the CSL to the inner Pier 46/48 slip.

Results of core dating analysis determined the approximate rate of sedimentation in the study area. The mean sedimentation rate was 0.28 $g/cm^2/yr$, not nearly enough to account for the higher rates of accumulation found in the traps (0.78 - 1.2 $g/cm^2/yr$). The additional material in the traps is likely a combination of two factors, resuspension of bottom sediments and natural organic matter in the water column from increased productivity in spring and summer months. The rate of recontamination of a cap or other clean area from resuspension of nearby contaminated bottom sediments cannot be easily calculated, since it will depend on the size of the clean area relative to the size of the contaminated area, the proximity of the remaining contaminated sediments to the site, the degree of contamination remaining, and the frequency and mechanisms of resuspension in the area. These factors were addressed later in the study.

Identify the components of recontamination and quantify the contribution of each component to the extent possible. Possible sources of contaminated sediments in the traps, and thus potential sources of recontamination, include ongoing point discharges (outfalls), nonpoint sources, and resuspension of contaminated bottom sediments. As discussed above, with the exception of the S. King St. CSO, maximum loading of contaminants from the outfalls and the Duwamish River was estimated and found to be a relatively insignificant source of contaminants to the study area. These sources are not likely to cause recontamination following cleanup.

Non-point sources, on the other hand, may have the potential to affect the area. Measurements of LPAHs in sediment traps were higher than concentrations of LPAHs in surface sediments. This suggests that there are ongoing sources of LPAHs other than (or in addition to) the minor amounts of PAHs discharged through the outfalls. Potential non-point sources of LPAHs include small spills of fuel, discharges of oily water from vessels, and creosoted pilings and bulkheads, particularly those in disrepair and potentially decomposing. These sources are ubiquitous along the waterfront and not likely to be easily controlled.

Although concentrations of LPAHs in some of the traps exceeded CSLs, measured concentrations of LPAHs in surface sediments in the most of the study area are below CSLs. The exception is the area adjacent to the northern end of Pier 52. This area was apparently contaminated by a remnant subsurface source of LPAH material, high in phenanthrene, which was unearthed during the wing wall removal (KCDMS, 1994a).

The lack of widespread LPAH surface sediment contamination over most of the study area is consistent with LPAHs' relatively short half-life in aquatic environments. This suggests that, in spite of continuing non-point sources, LPAHs are apparently not building up in surface sediments to levels that would exceed CSLs. However, the Panel would be advised to select a less stringent site-specific cleanup standard, such as the CSL, for this class of contaminants to allow for the likelihood that some recontamination with LPAHs will occur.

Except for LPAHs, the other contaminants found in sediment traps are likely related to resuspension of contaminated bottom sediments. Patterns of contamination are very similar to those in bottom sediments, and other sources do not appear to be present. The mechanism of resuspension appears to be primarily prop wash from vessel traffic. This finding is supported by the fact that accumulation of contaminated sediments in traps is highest in areas with the most vessel traffic, and in seasons (spring to late summer) when the frequency of vessel traffic is highest. For nearly all the contaminants, this source of recontamination is considered the most significant, and accounts for the CSL exceedences observed in the sediment traps.

Current meters placed along the waterfront measured the speed and direction of natural currents in the study area. These currents are generally below 5 cm/sec, not high enough to resuspend any but the most fine-grained sediment. Modeling of wind waves and wakes showed that waves were likely to resuspend sediments only in a narrow band along the shoreline. Current meter records also showed "spikes" - short-term higher velocity events, that generally occurred in areas

of high vessel traffic. A subset of these high-velocity events were checked against vessel traffic records, and were nearly all assignable to movements of specific vessels in the area. Modeling of a variety of vessels along the waterfront confirmed that many of the vessels currently operating along the waterfront have the ability to resuspend fine-grained sediments, particularly when maneuvering, docking, or idling in place. As has been demonstrated at Colman Dock and Pier 65/66, waterfront construction activities also have the potential to resuspend and/or expose contaminated sediments.

If the rate of recontamination is unacceptable, identify source control and/or resuspension control measures that would reduce recontamination to an acceptable rate. Source control recommendations have been developed to address each of the sources of recontamination discussed above. Although none of the storm drains along the waterfront is believed to be great enough to cause significant recontamination, the S. King St. CSO is the largest of the remaining discharges and is likely to cause localized elevation of concentrations in sediments near the outfall. If source control funds remain upon completion of Norfolk and Duwamish/Diagonal sites, the King St. drainage basin may benefit from similar source control activities, such as removal of sediment in lines and catch basins, inspections, education of businesses on BMPs, and identification and elimination of any remaining improper discharges. However, cleanup of the slip between Piers 46 and 48 may need to be delayed until source separation of the King St. CSO is accomplished.

Non-point sources of LPAHs will be difficult, if not impossible, to control. Reduction in these sources over time is expected as spills and discharges from vessels and refueling are better controlled, creosoted pilings are replaced with non-polluting alternatives, and point discharges are further controlled. Cleanup of surface sediments should eliminate the contribution from existing bottom sediments. Because the highest levels of LPAHs in surface sediments are present in the vicinity of Colman Dock, careful coordination of Panel cleanup activities with Colman Dock cleanup activities is also advised and should reduce the potential for future recontamination by LPAHs.

Careful control of construction activities is also recommended to prevent exposure and resuspension of buried contaminants that exist at depth along nearly the entire Seattle Waterfront. BMPs for pile pulling and other construction activities are currently under development by an interagency workgroup led by DOT, Ecology, and the Port of Seattle. The Panel should carefully consider how its cleanup activities may affect or be affected by planned waterfront construction activities, such as Port development, Colman Dock construction plans, and aquarium expansion plans.

Finally, controlling resuspension of contaminated bottom sediments is critical to the success of any waterfront cleanup project. Because resuspension is primarily related to vessel traffic and nearshore wave action, control of this problem is directly related to the design and selection of remedial alternatives. The size and location of cleanup projects with respect to currents and vessel uses is important, as is selection of appropriate capping materials.

Once resuspended, contaminated fine-grained sediments will be transported along the waterfront by natural and induced currents. The current meter data collected during this study identified two areas of general circulation. Currents along the waterfront appear to be affected by the large automobile ferries idling at Colman Dock, and converge from the north and south on the ferry terminal, where water is moved offshore. In addition, there are nearshore circular eddies in relatively open areas between piers. This circulation pattern indicates that the central waterfront can be separated into two general areas, one north and one south of Colman Dock, that can be remediated independently because it does not appear that suspended sediment would easily travel from one area to the other. Each of these areas should be remediated in its entirety, since contaminated sediments resuspended in one location within the area could be transported to any other location within that area.

Model the impact of these recontamination processes on potential sediment remediation options for the study area. Based on the source control evaluation, a detailed model of discharges along the waterfront was not necessary, due to the low potential for these discharges to result in recontamination. There is no known model that can readily predict the impact of non-point sources of LPAHs on the waterfront. However, the worst-case possibility was evaluated above by looking at existing concentrations in surface sediments, since sources of LPAHs are expected to decline in the future. The third major source, sediment resuspension, appears most directly related to vessel prop wash. This process could have a direct effect on the success of remedial alternatives such as capping. Therefore, the next phase of the study was to model the potential effect of prop wash from existing vessel traffic on the Seattle Waterfront on a standard cap design.

The scour potential modeling suggests that some scour may occur in a 3-foot thick remediation cap of sand placed along the Seattle Waterfront in at depths of less than about 40 feet. However, the predicted depth of scour does not seem to be serious enough to warrant initial placement of a gravel armor layer. Periodic monitoring of the remediation cap is recommended.

To further support the Panel's remedial design efforts, information has been provided on sources and costs of capping material of various types, and on remedial alternatives and costs for cleanup under piers.

Based on all of the above, provide the Panel recommendations on whether cleanup along the waterfront is feasible, the most appropriate project location(s) for sediment remediation, and the size and type of project(s) that would have the greatest chance of success. Based on the study results presented above, the study team believes that, with the exception of the S. King St. CSO, ongoing sources are adequately controlled and remediation along the waterfront is possible at this time. However, due to the potential for resuspension of contaminated bottom sediments, areas should be selected with current circulation patterns and vessel traffic patterns in mind. The team recommends two large cleanup areas, one extending from Pier 46 to the south end of Colman Dock, and one extending from the north end of Colman Dock to Pier 59. Cleanup of the north area should be fully coordinated with the cleanup of Colman Dock to minimize the

potential for recontamination of both projects. In addition, coordination with the aquarium expansion project to the north is also recommended. Cleanup of the south area should be coordinated with source separation of the S. King St. CSO.

Based on scour modeling, capping should be a feasible alternative in the north area, given current vessel use. In the south area, caps should be monitored periodically for scour in areas of active vessel use, and to determine if armoring may be required. Major changes in pier configuration and/or use of an area by larger vessels in the future may necessitate dredging or armoring of caps at that time. Dredging could also be considered in areas where scour is expected in order to maintain navigation depth, and to avoid potentially resuspending contaminated material.

Readers of this report should bear in mind that this study was conducted for the specific purpose of providing the Panel with cleanup recommendations for the Seattle Waterfront. It was outside the scope of this study to fully evaluate the sources of historical contamination, liability for existing contamination or cleanup, or the potential for natural recovery of sediment contamination. The nature of this study is highly site-specific, and its conclusions should not be generalized and applied to other areas that may have differing environmental conditions and aquatic land uses.

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